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IMPROVED DYNAMIC MODELLING FOR DRAINAGE SEDIMENT PREDICTION AND MANAGEMENT

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I certify that this thesis is the true and accurate version of the thesis approved by the examiners.

Signed .[redacted]... Date........08/08/2005

Abstract

The study reported in this thesis aims to further knowledge of sediment behaviour and provides engineers with appropriate tools and methods to facilitate proactive sediment management. The techniques developed during this investigation are therefore able to predict the locations and quantities of sediment deposits through improved sediment transport methods and offer a strategy of sediment control using sediment traps where appropriate. To facilitate these methods, a number of the previous shortcomings of drainage sediment modelling required to be addressed. Most notably these included: single particle size limitation, impractical processing times; the use of purely granular analysis; and no feedback between sediment and hydraulic models.

A programme of data collection was devised to develop and test the techniques of this investigation, focussing on sediment processes in sewer pipes and sediment traps. The data collected included, suspended and near bed transport rates, long-term sediment bed development and sediment trap filling patterns. These data and previous data sets were then used in the development of analytical methods for the prediction of sediment behaviour. Field tests were also undertaken to assess the performance of an alternative form of sediment trap using partial covers. These tests highlighted the importance of good site selection for sediment traps as no improvement in trap performance could be achieved at a poor trap location through the alteration of trap form.

The resulting sediment transport models were typically developed using a combination of the historical and new short-term data sets, and were then verified using newly collected long-term data. Models were developed for rapid hydraulic simulation, sediment location prediction, sediment erosion, sediment movement, sediment deposition and trap filling. The current limitations of sediment modelling were addressed through a number of innovations which have built on the experience of CIRIA report 141. Following the development of each component model, an overall sediment prediction model was created in the SIMULINK programming

environment. This combined model took the interdependency of the various sub-models into account and represented interacting processes where appropriate. The full model was then tested against a new long-term data set and was then applied to a new catchment to demonstrate suitability as a sediment management tool. The implementation of the various sediment transport methods developed during this study have not only allowed the identified limitations to be overcome but have also led to the development of a model with reduced likelihood of inappropriate calibration.

In verification tests, the combined sediment prediction model was found to be able to predict the location of sediment deposits to a level of accuracy suitable for allowing operational decisions to be made. Under quantitative tests, the deposition model was found to predict depths of sediment deposits to within -5 and +10%. A high level of accuracy was also achieved in the prediction of sediment trap fill rates at three test sites. However, in each case some local calibration of the model was required.

Analysis of the behaviour of the sediment deposition model and observed data has informed the identification of influencing factors in bed development. These influencing factors of pipe gradient, initial deposit location, bed gradient reduction, increased roughness and then finally increased bed gradient, result in the formation of an "S" curve reaching an equilibrium sediment level. The erosion model tests have indicated that for practical reasons, the concept of critical erosion criteria should be re-assessed as a result of the deposition that typically follows an erosion event. It is proposed that a new critical erosion criteria should be applied that allows for a net erosion during rainfall events

These analyses have furthered knowledge of sediment behaviour and combined with the modelling techniques devised during this investigation, can be used to confidently predict sediment behaviour in drainage systems and enable proactive sediment management.

Acknowledgements

The details of my work whilst at the University of Abertay Dundee are described extensively within this thesis. However, this only tells part of the story. During my time at the Urban Water Technology Centre, a large number of people have contributed in various ways to the completion of this work and ensuring that I could not have enjoyed my period as a researcher any more than I did. The following is therefore an attempt to thank those who have helped during these times:

The Engineering and Physical Sciences Research Council (EPSRC) and Scottish Water (formerly North of Scotland Water Authority) for the financial support given to this project. In the case of Scottish Water, further appreciation is extended to Mr P. MacKay and Dr D. Wotherspoon for allowing and arranging access to sewer sites and support staff, and allowing the research team to influence operational activities.

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Notation

A	Cross-sectional area of flow
a	Rate of surface sediment supply
A_b	Cross-sectional area of sediment bed
a^*	Reference concentration level
b	Surface sediment removal constant
C_{a*}	Sediment concentration at reference level a*
Cv	Volumetric concentration
C_y	Sediment concentration at level y
D	Pipe diameter
d_{50}	Median sediment particle diameter
D_{gr}	Ackers and White dimensionless grain parameter
D_r	Total depth of last rainfall event
D_*	Dimensionless particle size
e	Void ratio
e_b	Sediment trap bedload retention efficiency
e_{sus}	Sediment trap suspended solids retention efficiency
e_t	Overall trap efficiency
Frd^2	Shields' entrainment function
f	Darcy-Weisbach friction factor
$f(t)$	Pollutant flow
g	Acceleration due to gravity
H_e	Erodible sediment depth (Wotherspoon model)
H_o	Initial average bed depth
I_r	Peak intensity of previous storm
i	Bed gradient
J_m	Mean jump length
J_{max}	Maximum jump length
K	Linear reservoir coefficient (from Desbordes)

Ka	Erosion / dissolution factor related to rainfall intensity
k_b	Equivalent bed roughness
k_s	Equivalent sand roughness
k_w	Equivalent wall roughness
L	Length
m	Water content
Me	Mass of sediment eroded
Me(t)	Mass of sediment in suspension
M(t)	Mass of surface deposits
n	Manning's coefficient
P_b	Bed wetted perimeter
P_w	Wall wetted perimeter
Q	Volumetric discharge
Q_b	Bedload flux rate
Q_{fill}	Sediment trap fill rate
Q_s	Sediment flux rate
Q_{sus}	Suspended solids flux rate
R	Mean hydraulic radius
Re_*	Reynolds number at grain
s	Pipe slope
SG	Specific gravity
S_s	Sediment specific gravity
T	Shear ratio
TL	Trap length
TSSS	Time since start of last storm
u_*	Shear velocity
W_b	Sediment bed width
w_s	Particle settling velocity
X	Sediment surface load
y	Flow depth or level above pipe invert
y_b	Sediment bed depth
y_{max}	Daily peak dry weather flow depth

y_o	Ambient flow depth or level above pipe invert
Z	Percentage of dry weather solids deposited
Z_{gr}	Relative grain size
Φb	Einstein's transport parameter
η	Sedimentation parameter
κ	Von Karmen constant
λ	Friction coefficient
λ_b	Bed friction coefficient
μ	Dynamic viscosity
ν	Kinematic viscosity
Θg	Einstein's grain mobility
ρ	Fluid density
ρ_d	Dry density of bulk bed material
ρ_s	Sediment density
ρ_w	Wet density of bulk bed material
$\overline{\rho}_o$	average initial bed density
τ_b	Bed shear stress
τ_c	Critical boundary shear stress
τ_o	Average boundary shear stress
τ_y	Yield shear strength
ξ	Density coefficient
ζ	Erodibility coefficient

Chapter 1 : Introduction

1.1 Historical Background

The primary method of dealing with sanitary waste and storm runoff in the developed world is the provision of underground sewerage networks. These networks must transport both liquids and solids to points of treatment or disposal. This form of sewerage system has been found to date from approximately 1700 BC (McGhee, 1991). The Minoan Palace of Knossos on the isle of Crete featured four terracotta pipe systems that emptied into great sewers constructed of stone. However, traditional underground sewerage was not brought to the UK until the advent of the Roman Empire. Following the demise of the Roman occupation, drainage and sewerage systems were neglected until public health became a national issue in the early 19th Century. Following frequent outbreaks of cholera and other diseases, a government investigation was instigated. The resulting report of Edwin Chadwick on "*The Sanitary Condition of the Labouring Classes*" (1842) was central in evaluating the role of sewers in the public health of the nation. Victorian engineers were acutely aware of the potential problems of solids depositing within drainage systems. Chadwick's enquiry revealed details of how deposits were managed.

"*... the streets were opened at a great expense and obstruction ....; men descend, scoop up the deposit into pails, which are raised by a windlass to the surface, and laid there until the carts come; it is laid there until it is carted away, sometimes for several hours, to the public annoyance and prejudice. The contract price for removal from the old sewers without manholes was 11s. per cubic yard of slop removed; where they have manholes it was 6s.10d. per cubic yard.*"

As a consequence of the difficulties encountered in removing the deposits, they were often allowed to remain in pipes for 5 to 10 years. Following the Chadwick report, Victorian engineers undertook a massive rebuilding and maintenance programme which forms the general model for sewerage design and management today. Steeper gradients were introduced to encourage higher flow velocities and hence higher sediment transport capacities. The original flat-bottomed sewers were also replaced with curved egg-shaped or circular conduits. The debate surrounding the most efficient section for transporting solids continues today, with some ironically suggesting a rectangular section (Loveless, 1991; Torfs, 1994). Other innovations in the area of sewer system management followed but were often pioneered by individual drainage engineers. These included sewer flushing and the installation of sediment traps.

Following Chadwick's enquiry, it was recorded that Mr. John Roe, Civil Engineer to the Holborn and Finsbury Commission of Sewers, adopted a maintenance strategy of sewer flushing. The cost of sewer cleaning in Holborn and Finsbury was consequently reduced from £12,000 per year to £600 (Sellers, 1997). Flushing tanks were also included in the design of the sewerage system of Dundee by the consultant engineer Mr John Frederic Bateman, Civil Engineer at London and Manchester. An additional design feature of the Dundee system, which makes it unique within the UK, is the provision of a network of sediment traps. Recent surveys have however also revealed the use of traps in other catchments at specific locations (Fraser et al., 1998). The purpose of the network of Dundee traps was stated in an article appearing in the Dundee Advertiser on 21st January 1884.

"*The sand trap is intended to catch all heavy substances carried along in the sewage, which would in all probability deposit in and choke the outfall. These sand traps are cleaned out about once a fortnight in summer and once a month in winter.*"

Although many of the Victorian innovations were widely adopted by practicing engineers, the wide use of sediment traps was not. It is believed that this was a result of the labour intensive nature of the cleaning processes and the inability to predict and plan cost effective maintenance (Fraser et al, 1998). Notwithstanding these difficulties, it has been apparent from Victorian experience and more recent operational experience in Dundee and France that the opportunity exists to reduce operational budgets by localising sedimentation to designated zones where large sediment volumes can be stored and easy access facilitated (Paitry et al., 1990).

Subsequent advances in the area of sewerage design and operation have until recently concentrated on strengthening sewer structure with improved materials and renovation technologies (Ashley et al., 2001). A large void in the knowledge of operational processes of sewers therefore existed until the early 1980's.

During the 1980's, sewerage undertakers realised the enormous financial asset represented by the traditional underground drainage networks. Consequently, sewerage investment policies in recent times have tended towards increased capital expenditure rather than maximising operational aspects. This has led to a general ignoring of the development of improved operational practices. However, in recent years, increased urbanisation, growing populations and climatic change are placing greater strains on existing sewerage networks, leading to systems which are undersized and inefficient in dealing with storm water runoff (Xanthopoulos & Hahn, 1994). Due to the vast capital expenditure involved in the upgrading of a typical sewer system, it is the contention of many researchers today that it is preferential that existing systems should be 'optimised' to bring about the maximum level of performance at the lowest financial cost (Ashley et al, 2004; FWR, 1994).

As a consequence of a Europe-wide policy for the protection of receiving waters and a substantial increase in the environmental awareness of the general public, a concerted group of research initiatives were instigated on sewer processes in the early 1980's. The majority of this work emanated from Northern Europe, with the UK playing a key role. One of the most important conclusions to emerge from this work was the impact of sewer and drainage sediments on both physical system behaviour and operational costs. The cost in the UK of managing sewer sediments at this time was put at approximately £50-60m per annum (CIRIA, 1987).

Many research projects since this initial work have concentrated on the movement of sediment in sewerage systems and the physical effects of sediment deposits. Importantly, many of the techniques traditionally applied to river sediment transport have now been modified and tested for drainage sediments along with completely new drainage sediment studies. This has enabled estimates of sewer sediment transport rates to be made and although catchment-wide testing of sediment transport models has been rare, the opportunity to apply these methods from a strategic maintenance point of view undoubtedly exists. The first real attempt to encourage drainage practitioners to employ the most appropriate techniques of sediment prediction was CIRIA Report 141 – Design of Sewers to Control Sediment Problems (Ackers et al., 1996). This report brought together and assessed, for the first time, a range of sediment prediction methods. The methods were presented in a framework to assist in the design of sewers and minimise sediment deposition in these designs.

The presence of sediment deposits is well known to limit the hydraulic capacity of pipes, and their contribution as a primary source of pollutants during wet weather is now widely acknowledged although poorly understood (Ashley et al., 1994). It has also been proposed that important problems of sediment deposits and flushed pollutants arise from a dense 'semi-fluid' layer moving as 'near-bed' solids close to the pipe invert or sediment bed (Ashley & Verbanck, 1996). The polluting potential of this material is significant (Arthur, 1996, McGregor et al, 1997) and these pollutants, together with deposited sediment may be re-entrained during accelerating flows in a "foul flush". Surrogate laboratory experiments also support the existence of such flushes (Skipworth, 1996). As well as pollutant shock loads at combined sewer overflows (CSO's) and wastewater treatment plants (WWTP's), other operational difficulties occur due to the nature of the solids involved. The larger particles can result in screens and grit lines choking, whilst smaller grit can cause the rapid wear to the teeth of comminutors. For example, in Aberdeen (Scotland) hard stones (circa 25mm in size, known as 'chuckies') are conveyed to the WWTP from the erosion of the rock tunnel walls in the larger sewers, causing screen damage. In addition to this, organic gross solids, a major component of the near bed solids load, have been frequently noted to blind drum screens (Ashley et al., 1995).

As a result of the diversity of solids experienced within combined sewer networks, and the range of possible solids ingress paths, the exclusion of sediment from sewerage systems is at present, highly unlikely. This, coupled with the high costs incurred by the periodic cleaning of sewer lengths affected by sediment deposits suggests that alternative or enhanced approaches to managing sediment should be developed.

1.2 Aims and Objectives

Solids accumulated in drainage systems are known to create a number of hydraulic, operational and environmental problems. However, current drainage management strategies offer only reactive maintenance; typically dealing with the cause of a problem after it has occurred. Since the early developments of sewerage management between 1850 and 1900, only modest advances have been made in sewerage design and methods for managing solids. The principal advances made in the studies of drainage sediments have come in the last decade. As a consequence of these studies, the point is now being reached where previous experiences can be drawn together and applied to solve practical, sediment related, drainage problems. To date, attempts to apply sediment transport prediction methods using commercial software have proved problematic as a result of data collection and calibration difficulties.

The aim of the research reported in this thesis is to further knowledge of sediment processes and provide engineers with readily applied numerical techniques for the management of drainage sediments. In order to achieve this aim, the objectives were:

- Assess the shortcomings of current sediment management prediction methods and practices through a review of relevant literature;
- Overcome any identified shortcomings through the modification or addition of numerical methods. The methods should be able to perform the following operations:

a) Predict sediment transport rates using readily available data and reduce the potential for model "force-fitting";
b) Predict potential locations of problematic sediment deposits;
c) Predict quantities of deposits at these locations allowing for deposition and erosion processes;
d) Provide an analytical model for a method of sediment control. The method of control selected within this study was the use of sediment traps;
e) Combine model elements into a dynamic interacting sediment model.

- Apply and assess methods using an alternative drainage system;
- Define an applicable proactive sediment management strategy.

1.3 Study Methodology

Figure 1.1 shows the overall strategy undertaken to complete the study. Following the establishment of the study's aim and objectives, a literature review was carried out in order to ascertain the current level of knowledge and to inform the other phases of the study. This allowed the successful planning of data collection and modelling activities.

The data collection activities required an initial desk study collating the data from previous studies, development of data collection methods in the laboratory and their subsequent application to field sites. These activities focussed on the collection of rainfall, hydraulic, sediment transport, sediment deposit and trapped sediment data at a range of sites around the Dundee area. These data were also used to plan and later refine modelling methodologies.

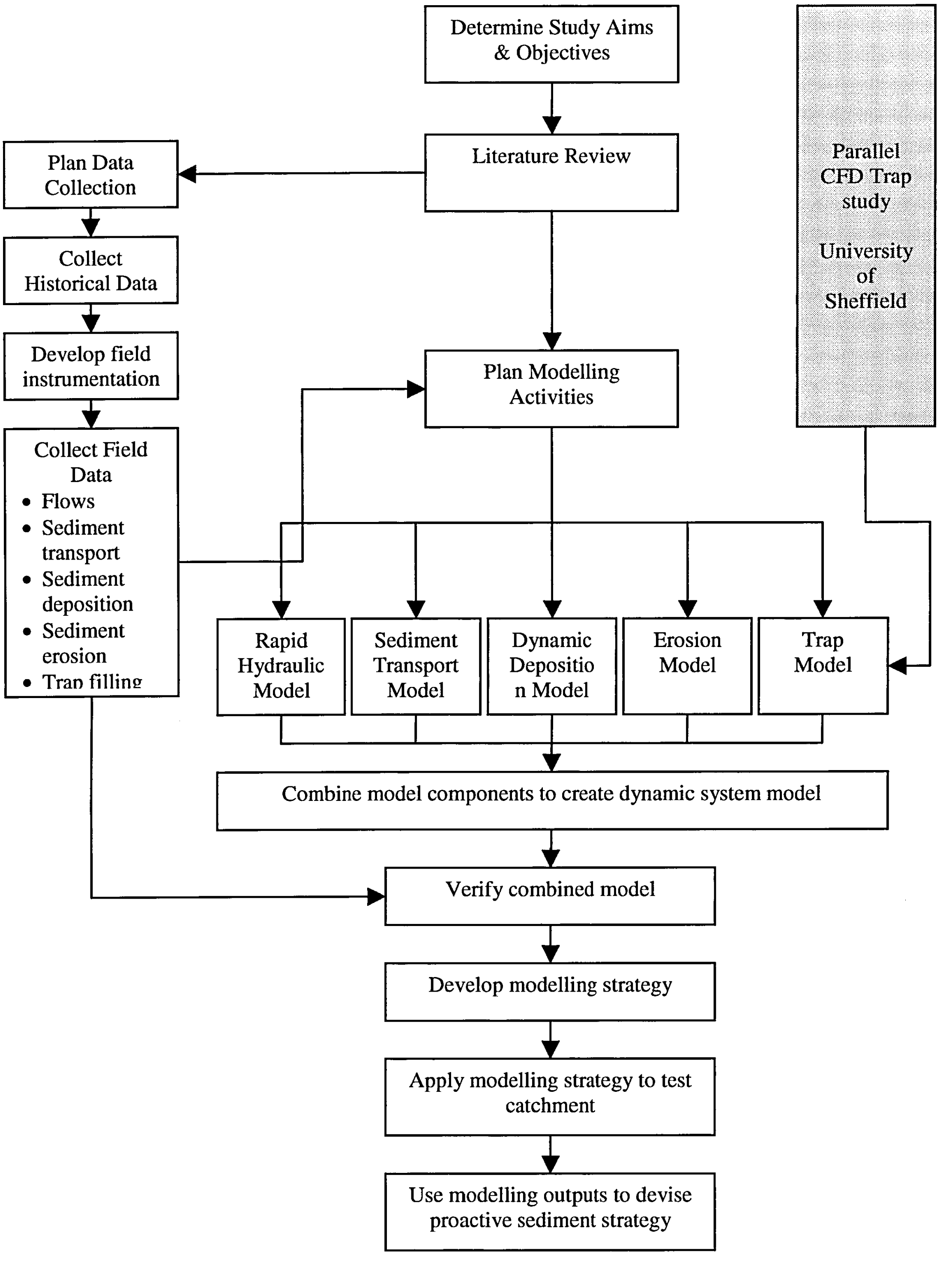


Figure 1.1 - Project overview

The modelling activities were required to provide an appropriate level of performance to facilitate proactive sediment management. The prediction of sediment behaviour required the modelling of sewer system hydraulics, sediment transport, sediment deposition and sediment erosion. In addition to this, in order to provide sediment management strategies to be developed, a sediment trap model was included in the strategy. This particular method of sediment control was chosen as a result of the effectiveness of the method and the local availability of field test sites. In order to develop the trap model, an efficiency curve determined from Computational Fluid Dynamics modelling was used to establish the retention of suspended material. These CFD studies were carried out under a parallel programme of research carried out by Buxton (2004) at the University of Sheffield.

In general, models were developed and calibrated using historical data sets. This was done to allow verification to be undertaken with newly collected data. Where historical data were not available, new data sets were collected for calibration purposes.

Each component model was then combined with the others to produce a dynamic system model capable of not only modelling the individual component parts but also allowing these parts to interact as they do in reality. This combined model was then successfully verified against appropriate elements of the data collected during the field data collection phase.

A proposed strategy for the effective management of sediments was then developed using the combined model. Finally, this modelling strategy was tested and successfully applied to an entirely new catchment for the management of a real sediment problem.

1.4 Study Context

The work reported within this thesis is part of a wider investigation into the use of sediment traps in sewers for the management of sewer sediment related problems. The project is a collaborative study funded by EPSRC involving:

1. The University of Abertay Dundee – Principal researchers concerned with model development and full-scale field observation and testing.
2. The University of Sheffield – Physical and Computational Fluid Dynamics (CFD) modelling of detailed trap efficiency and design.
3. The University of Liverpool – Developers of simplified hydraulic models for long term computer simulation.

1.5 Thesis Structure

This thesis is constructed in seven chapters.

This chapter (Chapter 1) introduces the reader to the work and sets out a framework to guide the reader through the thesis.

Chapter 2 provides a review of previous and current research in the areas of sediment transport, sediment management and the application of sediment traps. This review is used to assess the requirements for research and direct the following chapters.

Chapter 3 details the field activities and data collection procedures used during the study. Descriptions of all of the sites monitored and the data provided by these sites are given along with an assessment of the data and the techniques employed.

Chapter 4 describes the development of each of the component parts of a sediment prediction model and details how these component parts should interact in order to best represent long-term sediment behaviour and minimise the problems currently associated with commercial drainage sediment modelling. The final combined model (Fraser sediment model) was tested against a long-term data set of pipe deposition and erosion.

Chapter 5 establishes a framework in which the Fraser sediment model can be applied to assist practicing engineers in the management of sediment related problems. This is done through applying the modelling methods to a test catchment exhibiting sediment related problems.

The principal findings of the programme of research are summarised in Chapter 6 along with recommendations for areas of further study.

Chapter 7 lists references used in the preparation of the thesis.

Chapter 2 : Literature Review

2.1 Introduction

This chapter summarises the existing literature relevant to this thesis and assesses the applicability of the methods investigated. As a result of the diversity of the areas investigated in this study, the review includes sections on sediment problems; sediment sources; characteristics; management practices; behaviour and modelling.

2.2 Drainage Sediment Related Operational Problems

Sediment related operational problems have been well documented in the UK since the middle of the 1980's (CIRIA, 1987). At this time a survey into the extent of sediment related problems was carried out by Hydraulic Research in association with the consultants Binnie and Partners (CIRIA, 1987). The report concluded that 80% of undertakers acknowledged the presence of sediment in their sewerage networks and that during storm periods 95% of systems are subjected to surcharging. Surprisingly, over 45% of these systems were found to surcharge during dry weather flows.

The most commonly observed problem is that of a reduced hydraulic capacity. Unfortunately, this problem is usually only identified as a result of localised flooding during high flows. In these cases the cost of any damage caused by the flooding or damage incurred within the pipe during pressure flows will often have to be met in addition to the cost of rectifying the sediment problem. Another problem of reduced hydraulic capacity can occur at combined sewer overflows (CSO's). The presence of sediment located downstream from a CSO can effectively reduce the hydraulic setting of the structure. This results in the earlier and more prolonged operation of the CSO during storm events.

Other operational problems associated with sediment deposits have been shown to include the production of gases and corrosive acids, in-pipe septicity problems and foul flush behaviour impacting control systems.

2.2.1 Conveyance Restrictions

The mechanisms and type of material involved in a capacity restriction can vary greatly depending on the size of the pipe. The type of materials and pulsed flow characteristics of small pipes (< 150 mm) gives rise to the potential for the accumulation of faeces, toilet paper and other sanitary items at locations of poor pipe fitting and manhole finishing (Lillywhite & Webster, 1976). These problems tend to be localised and are generally termed as "blockages". Their prediction on a wide scale would be impossible as a result on their dependency on small, unidentified defects in the sewer construction.

Large combined sewers are characterised by having a much more evenly distributed diurnal flow pattern. Rather than pulsed flow, a base flow comprising domestic, commercial, industrial and infiltration inputs exists at all times. Under normal operation, the forces exerted by this base flow should not allow sanitary and faecal solids to accumulate. Consequently it is larger, inorganic sediment particles which cause capacity related problems in larger sewers. These solids can be laid down during periods of low flow or during the recession of storm flows and have been shown to correspond with structural and hydraulic discontinuities, such as joints, gradient changes and junctions (Gerard & Chocat, 1998).

The presence of sediment in drainage conduits limits the hydraulic capacity in two ways. The cross-sectional area available to pass flow is reduced and the total roughness of the conduit is increased as a result of the presence of the undulating sediment deposit surface (Butler & Clark, 1995). Typically, the result of this increased roughness is the reduction of local flow velocities. It has been hypothesised that from these initial deposits, two possible scenarios may develop (Butler et al., 1996).

1. The reduction in velocity may cause a reduction in the sediment transport capacity leading to further deposition. Deposition will continue until the point where pressure flows are induced as a result of the cross-sectional area lost. If sufficient head is available, an equilibrium will develop between the incoming and outgoing sediment concentrations.
2. The presence of the sediment bed induces increased turbulence in the flow column leading to an increased sediment transport capacity. Thus a condition of equilibrium is reached much earlier than in condition 1 (above).

The above conditions have been observed both in the field and in laboratory tests (Laursen, 1956; May et al, 1989; Laplace, 1991). However, the above assessment of the possible scenarios does not explicitly include the potential gradient change as a result of sediment bed evolution. Studies of sediment bed development and sediment transport in a trunk sewer were carried out in Marseille, France (Laplace 1991). In these studies, the principal factor affecting the equilibrium sediment level was reported to be the gradient developed by the sediment bed. The sediment bed profile was measured over a period of 1000 days. In this time, deposits in the upstream section of pipe were seen to increase in depth with the downstream deposits remaining approximately constant.

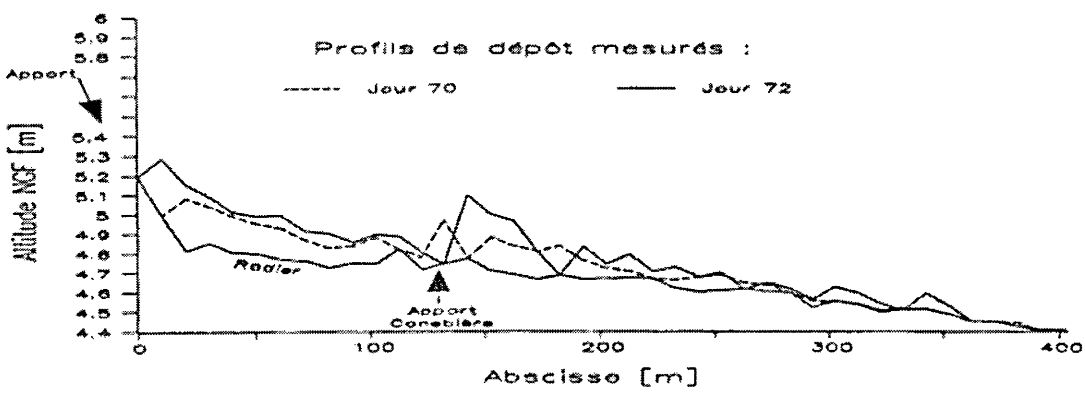


Figure 2.1 - Marseille sewer sediment profile, days 70 & 79 (Laplace, 1991)

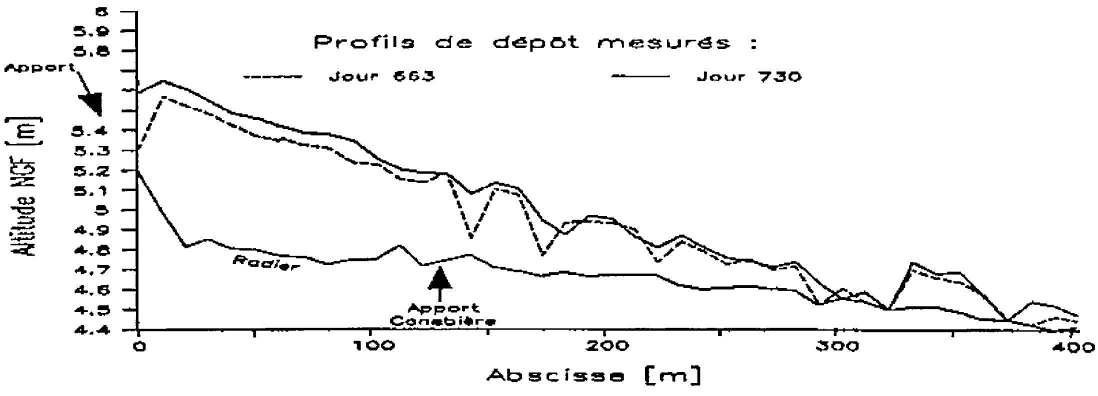


Figure 2.3 - Marseille sewer sediment profile, days 663 & 730 (Laplace, 1991)

Figure 2.1 and Figure 2.3 (above) demonstrate the change in gradient that took place in this study over the period from day 70 to day 730. Over this duration, the gradient increases from 1:575 to approximately 1:333. Under the assumption of steady, uniform flow, this has the effect of increasing bed shear stresses by a factor of 1.7. Although this is clearly a large difference in the normal operation of the pipe in question, at the present time this effect is generally ignored by analysts and drainage modellers.

The effects of even relatively low levels of sedimentation on hydraulic performance can also be significant. Field measurements in Sweden have indicated that for levels of sediment deposition in the order of 20% of the pipe diameter, a reduction of up to 50% of hydraulic capacity results (Perrusquía, 1988). A similar level of effect was found in the 1987 CIRIA study of UK drainage sediment movement (CIRIA, 1987).

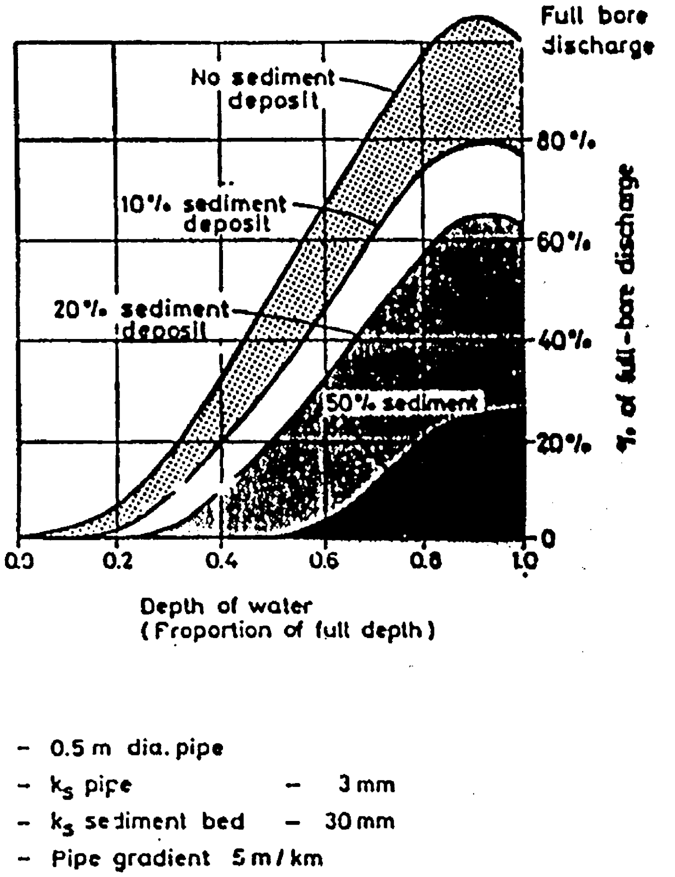


Figure 2.5 - Effect of deposition levels on hydraulic capacity (Ackers et al., 1996)

Figure 2.5 (above) shows the effect of various levels of sedimentation on discharge for various flow depths. For example, for the Murraygate sewer in this study, a sedimentation value of 20%, at a proportional flow depth of 0.6, corresponds to a flow reduction of around 40%. This compares well to the values determined in the Perrusquía (1988) study although proportional depths and discharges for these investigations are not known.

Other ancillary structures such as storage chambers, overflows and pumps are also affected by sediment deposition to varying extents. The design of storage tanks and overflow chambers in the UK aims to attain a "self-cleansing" condition. Although in reality the diversity of material encountered makes this difficult, few cases of capacity reduction as a result of sedimentation problems in tanks have been reported.

Studies into the design of chambers to maintain sediment free conditions are continuing using commercially available computational fluid dynamics packages (Kluck, 1997; Stovin, 1996).

2.2.2 Gas and Acid Production

The substances produced during the various transformation processes within sewer sediments can themselves create operational problems. Typically these problems centre on the production of gases and acids which can cause odours or attack the fabric of the sewer conduit. The production of gases often causes complaints from the general public as the principal gases produced (hydrogen sulphide and methane) are generally malodorous (Ashley et al, 2004). There is a dearth of information on the transformation processes which produce these gases and other volatile organic compounds (VOC's); therefore at present, the best way of eliminating these problems is through the minimisation of sediment deposits and ensuring that stagnant or septic conditions do not occur.

In the USA, VOC's must be controlled under the Clean Air Act (1990). Odours regularly escape to the atmosphere from the sewer via gullies, manhole covers and vents. Under normal operating conditions in the UK, odours are rarely a problem. However, in warmer climates with low dry weather flow conditions, odours are more noticeable.

The production of hydrogen sulphide (H_2S) can also lead to the corrosion of the sewer fabric itself. Given the correct conditions (slow moving flow, low re-aeration and high temperatures) hydrogen sulphide is released to the atmosphere within the sewer (Hvitved-Jacobsen et al, 1988). Following absorption onto the damp walls of the conduit, aerobic bacteria act on the hydrogen sulphide to produce sulphuric acid. This acid then attacks the concrete surface of the sewer wall (Hvitved-Jacobsen et al, 1988), as illustrated in Figure 2.7.

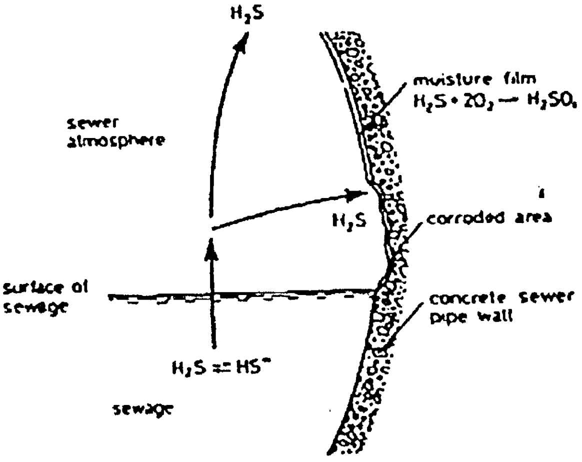


Figure 2.7 - Hydrogen sulphide release and corrosion

2.2.3 Foul Flushing Behaviour

Storm runoff from urban catchments typically has a rapid response causing highly peaked hydrographs within sewerage systems. The increased flowrates, depth and velocity in pipes may cause an overall dilution of dry weather solids concentrations, or alternatively bring about an increase in solids concentrations at a given point in time. This temporary increase in solids concentrations has been termed a "flush". The relative position of the flush with respect to a storm hydrograph leads to the terms "first foul flush" and "second foul flush". Although it is widely accepted that the phenomenon of foul flush behaviour is widespread (Ashley & Verbanck, 1996), studies in France have questioned their occurrence (Saget et al, 1996) and the categorisation of flushes has been recently redefined in an attempt at standardisation (Ashley et al, 2004). This is because arguments over the existence of first flush phenomena largely centre on the method of definition rather than the physical processes involved. The criteria used in the French studies to define the flush are unlikely to be fulfilled as these are very prescriptive (see 2.2.3.1 below). The debate has also continued as a result of the difficulties involved in the measurement of flushes and their unpredictable nature. Measurements carried out in Germany over a long duration have indicated that even in the same sewer, different storm events may produce either a diluting or flushing response, with little apparent differentiation as to the cause.

2.2.3.1 Foul Flush Definition

Historically, the rise in solids concentrations associated with a flush has been attributed to a combination of the washing in of surface sediments and the scour of sediment deposits. However, the exact definition of the points of flush and dilution are still debated (Ashley & Verbanck, 1996). The first definition was developed by Krauth (1970) and compares solids loadings to hydraulic loadings. Other definitions have looked at relative concentrations rather than loads (Pearson et al, 1986). The later definition by Saget et al., proposed that a first flush only occurs when at least 80% of the pollution load are transferred in the first 30% of the flow volume. As a consequence of this strict definition, only one of the 197 events tested during the Saget study was classified as a first flush. However, this form of definition is useful, and in general the definition proposed by Bertrand-Krajewski et al. (1993) seems to be widely applicable. In this case a flush is defined when 50 % of the pollution load is transferred in the first 30% of the flow volume.

2.2.3.2 Flush Sources

The rise of Total Suspended Solids (TSS) concentrations above normal dry weather levels in a flush, suggests that during the initial stages of the flush wave, additional sediment sources contribute to TSS loadings. Traditional analyses suggest the sources to be washed-in surface sediments and the erosion of in-pipe deposits (Dauber and Novak, 1983; Krejci *et al.*, 1987; Bachoc, 1992; Gromaire-Mertz *et al.*, 1998).

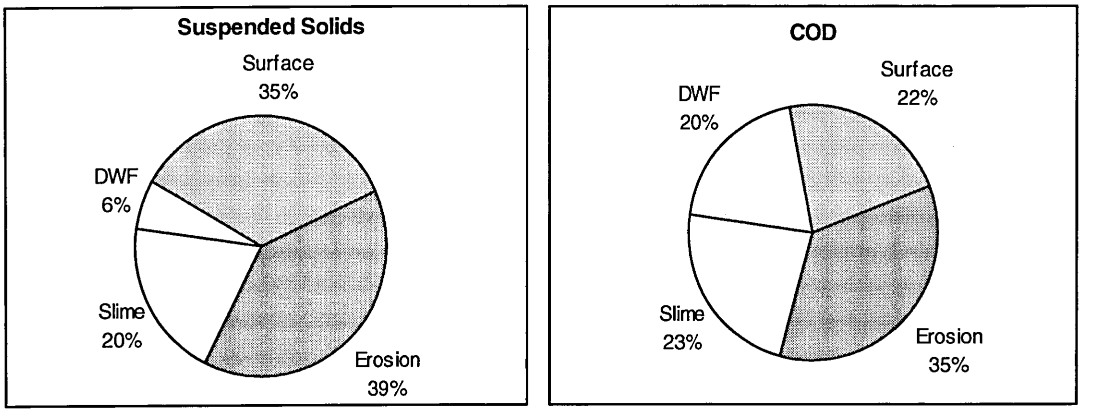


Figure 2.9 - Solids & pollutant sources in combined sewage flows (adapted from Krejci et al., 1987)

Figure 2.9 (above) shows results from a Swiss study. As can be seen the largest source in terms of both suspended material and chemical oxygen demand (COD) is that of previously deposited material. The most detailed studies to date of determining the sources of flushed material have been carried out in Paris (Gromaire-Mertz et al., 1998). Here, a small self-contained catchment was surveyed in detail, with flow, quality and rainfall measurements taken throughout the catchment over a prolonged duration. Heavy metal tracing was also used to try to track solids through the system. An extract of the results of this study are shown in Table 2.1.

	Contribution to load of solids in suspension (%)				
Rainfall (mm/h)	Sanitary Sewage	Roof Runoff	Yard Runoff	Street Runoff	Sewer Sediment
4.5	28	6	5	15	47
4.7	9	10	4	17	59
35.3	7	10	7	10	66
5.6	6	23	10	10	50
1.8	37	3	3	11	45

Table 2.1 - Storm solids sources (Gromaire-Mertz et al., 1998).

2.2.3.3 Foul Flush Problems

The flush of solids and associated pollutants can cause operational problems for treatment plants and ancillary structures.

Wastewater treatment plants are typically designed as continuous processes ideally requiring continuous inputs. If inputs vary significantly beyond the design parameters of the plant, problems may be encountered. During storm events, input flow rates and pollutant concentrations can vary greatly as a result of the foul flush and dilution effects.

In terms of physical treatment, the flush of deposits has been recorded accumulating on inlet screens (Kassner, 1987); blocking de-gritting devices; causing premature pump impeller abrasion; and reduced capacity in pipes, tanks and digesters. Biological processes are also affected, even by the presence of purely mineral material. The increase in mineral content is of high importance in the primary clarification stage of treatment, as increased volume of sludge may lead to the creation of anaerobic conditions. The varying organic content can cause an imbalance between biological growth and wastage of active biomass within biological processes (e.g. activated sludge).

2.3 Sediment Sources

As a consequence of the extreme variability of drainage sediment characteristics, attempts have been made to classify sediments based upon their origins. The principal sources have been identified but general rules for the relative importance of each source have not been established (Butler & Clark, 1995) as these would be site specific. Typically, five source sub-areas have been established:

- the atmosphere;
- catchment surfaces;
- domestic inputs;
- industrial inputs;

- below ground and sewer process sources.

The interaction of these sources and the various solids ingress paths are shown in Figure 2.11 (below).

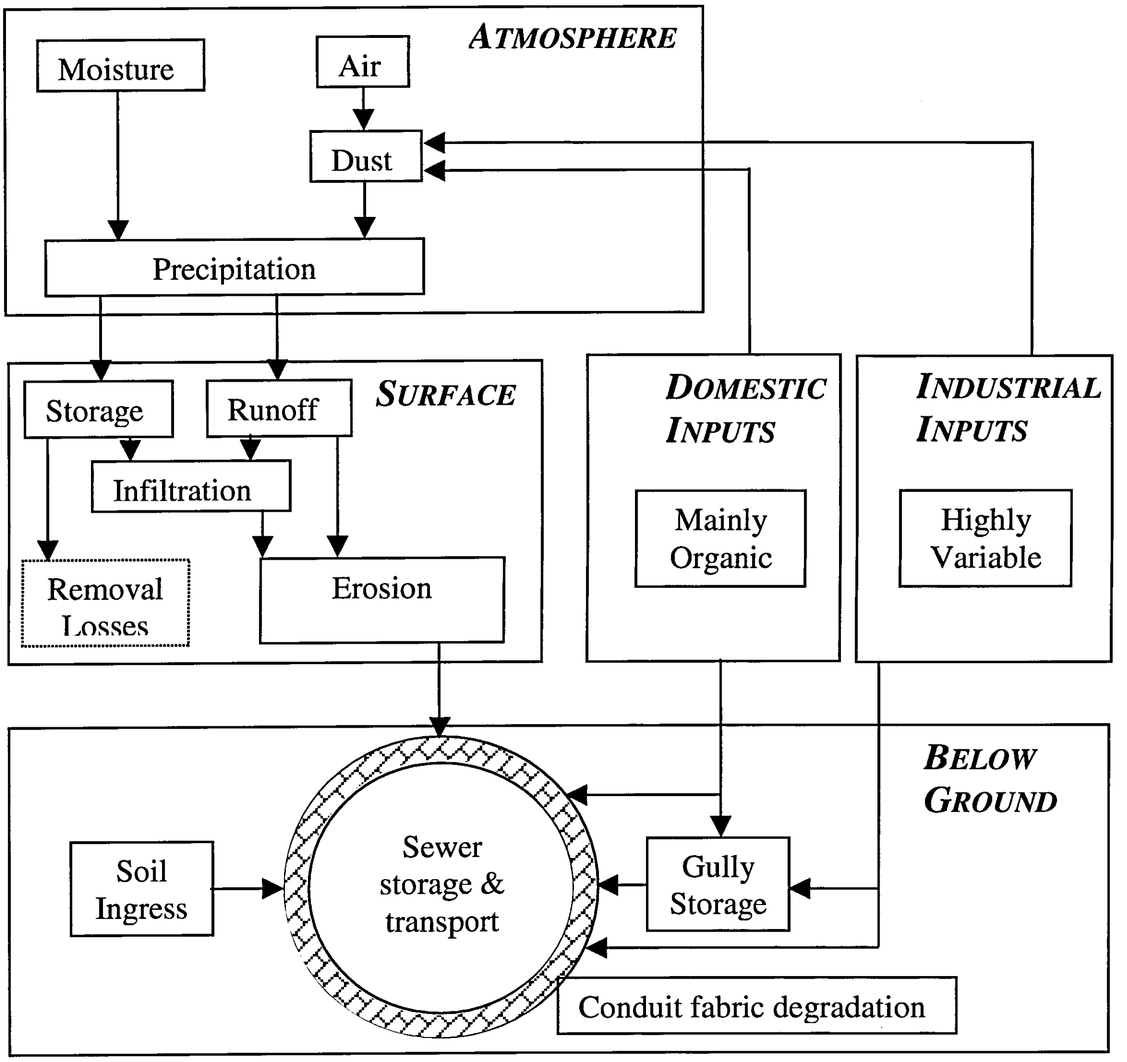


Figure 2.11 - Solids sources and ingress paths

2.3.1 Atmospheric Sources

Small sediment particles and aerosols are kept airborne by air turbulence and convection currents. These particles are released to the atmosphere from a wide variety of sources including the natural erosion of material and the release of pollutants from combustion and industrial processes. As ambient air temperatures cool, condensing moisture forms around these particles until a terminal mass is reached. The resulting falling raindrops therefore transfer the airborne particles to catchment surfaces. Studies have shown that the polluting potential of rain derived particle fall-out is unrelated to the characteristics of the rainfall transferring the pollution (Randal et al., 1978; Göttle, 1978). The sediment concentrations of rainwater have been shown to vary from 1 mg/l to 10 mg/l, with an average concentration of approximately 3.5 mg/l. Given the large volumes of rainfall required to initiate other sediment sources, the contribution of atmospheric sediments can be larger than expected, but is usually less than 10% of the total (Göttle, 1978; Artiéres, 1987; Uchimura et al., 1996). Pollutant concentrations of rainfall measured in a French study are given below (Table 2.2).

Pollutant	Min. Concentration	Max. Concentration
TSS	1 mg/l	10 mg/l
COD	20 mg/l	160 mg/l
Pb	1.6 µg/l	110 µg/l
Hydrocarbons	0.02 mg/l	0.07 mg/l

Table 2.2 - Pollutant concentrations in rainwater (Bertrand-Krajewski, 1993)

Even though the polluting potential for atmospheric solids can be high (Bertrand-Krajewski, 1993), the physical, chemical and biological impacts of this source are generally ignored in commercially available drainage modelling packages. The principal reason for this lies in the low level of understanding in the areas of the more significant sediment and pollutant sources.

2.3.2 Catchment Surfaces

A large number of studies has been carried out examining the role and physical characteristics of various catchment surfaces in contributing solids to sewers. Investigations in the UK in the 1980's revealed that catchment surfaces were the principal contributor to drainage sediment inputs (Ellis, 1986). The individual sediment sources that contribute to catchment surface build-up are extremely diverse and have been shown to include:

- road surfacing material and roadworks;
- winter de-icing operations;
- motor vehicles;
- washoff from adjacent areas (permeable and impermeable);
- construction work, stockpiles and spillages;
- industrial and commercial activity;
- litter;
- vegetation;
- roofs;
- atmospheric fall out (see Section 2.3.1)

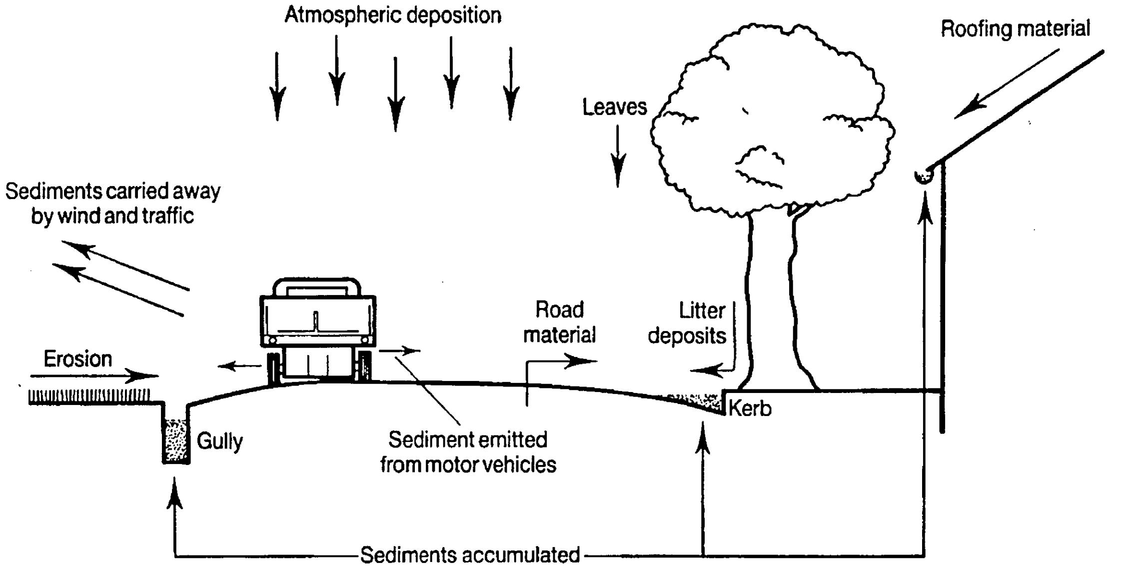


Figure 2.12 - Sources of surface sediment (from Butler & Clark, 1995)

Figure 2.12 (above) shows the diversity of sources, characteristics, transport routes and accumulation processes for surface sediments. As a consequence of this diversity and the temporal variation of each source (over various time scales), studies into their relative contribution have proved inconclusive. However, in the UK review of sewer sediments (CIRIA, 1987), a survey of sewerage undertakers was carried out in order to determine a rank of the importance of each sediment source. The results from this survey indicated that UK engineers perceived winter de-icing as the major source of solids entering the drainage system from surface sediments.

2.3.2.1 Winter De-Icing

The importance of winter de-icing as a source of solids for motorway surfaces was initially highlighted by Hedley and Lockley in a 1975 study. In addition to this, suspended solids concentrations during dry weather flows have been observed to be influenced by periods of road salting (Ashley and Crabtree, 1992). The potential influence of winter salting was highlighted by Butler and Clark using the following example (Butler & Clark, 1995):

Precautionary salting rate	10-15 g/m^2
Max. non-soluble content of rock salt	7.5 %
Salt solids loading rate	1 g/m^2
Typical surface sediment supply rate (excluding salt)	2 $g/m^2/day$

The values used in the example were determined from standard practice in the London area. Under the assumption that salt is applied every day, over every square metre of road surface, the contribution of salt solids to solids entering the sewer is 33%.

2.3.2.2 Vegetation

Another sediment source showing a significant seasonal variation is that of vegetation. During autumn, large quantities of organic material are deposited on catchment surfaces (roofs, pavements and permeable) as a result of falling leaves and vegetation residues. A USEPA report in 1973 (Heaney & Huber, 1973) found a typical leaf fall out from a forested area of approximately 9 kg/tree. An analysis of the material found it to be 90% organic and to contain between 0.04% to 0.28% of phosphorous.

2.3.2.3 Roofs

Traditionally roof drainage has been viewed as "clean". However, studies in the late seventies and early eighties attempted to quantify the pollution levels of roof runoff and determine important pollutant factors. The principal sources for these pollutants are:

- dry weather atmospheric deposits;
- degraded roof and gutter materials;
- vegetation (growing and deposited);
- bird droppings.

The dry weather deposition rates for these sources have been measured in many studies and have been found to vary greatly depending on local conditions (Ellis, 1986; Göttle, 1978; Förster, 1996; Sakakibara, 1996). However, UK experience has shown that the solids content of roof runoff contributes approximately 15% to 30% of the total solids load during rainfall.

Pollutant	Min. Concentration	Max. Concentration
TSS	0 mg/l	216 mg/l
Pb	10 μg/l	100 μg/l

Table 2.3 - Roof runoff pollutant concentrations

Table 2.3 (above) shows the range of concentrations for total suspended solids (TSS) and lead in previous roof runoff studies. In general, it was found that roofs in heavily trafficked or industrial areas experience the highest build-up rate of these pollutants. The studies of Förster (Förster, 1996) and Ellis (Ellis, 1986) have suggested that the contribution of these pollutants may be significant (especially heavy metals) in terms of a contribution to pollutant flushes

2.3.2.4 Pavement Surfaces

Much of the initial work on the build-up of sediments on pavement surfaces originated from the USA, with the majority of this research carried out in the 1970's as a prelude to the development of storm runoff models such as SWMM.

AREA	BUILD-UP RATE	UNIT
Residential	10-560	Kg/m of kerb/day
	95-3200	Kg/ha/yr
Commercial	13-180	Kg/m of kerb/day
	50-1220	Kg/ha/yr
Industrial	72-288	Kg/m of kerb/day
	400-1700	Kg/ha/yr

Table 2.4 - USEPA measured surface sediment build-up rates (Pisano et al., 1979)

The wide variation in build-up rates seen in Table 2.4 is due predominantly to the temporal variation for any particular site. In general, the accumulated mass of sediment on all types of catchment surface has followed an asymptotic pattern. An example of the type of pattern found in the studies of Pisano et al. (1979) and Huber and Dickinson (1992) is shown in Figure 2.13.

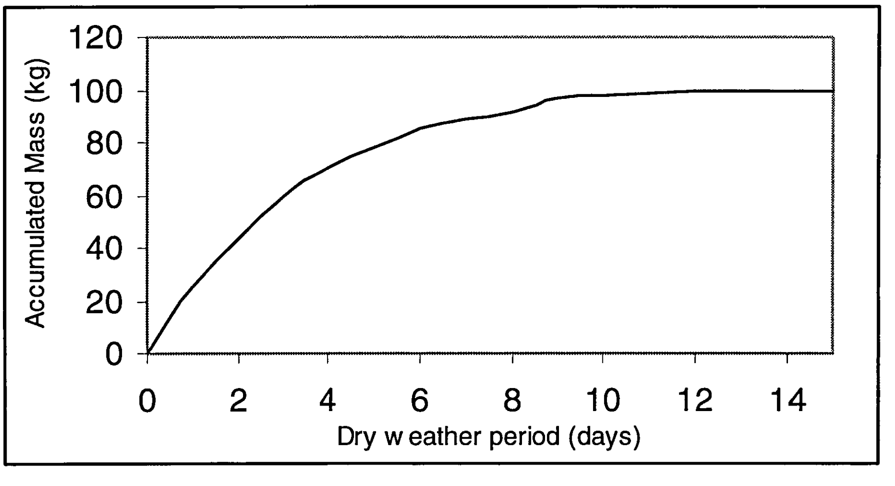


Figure 2.13 - Asympototic build-up pattern of catchment surface sediments (Pisano et al., 1979)

A number of relationships based on asymptotic functions have been developed (Huber & Dickinson, 1992). However, their application on a wide scale has provided mixed results; therefore careful consideration and site specific data collection are required for their use (Huber & Dickinson, 1992).

The most detailed UK study was carried out by CIRIA (Butler & Clark, 1995), examining various catchment types in the London area. These data were then used in the development of UK sediment build-up models. The resulting model (Equation 2-1) varied from the US studies, as although an asymptotic pattern is formed, a linear deposition pattern is assumed during periods of dry weather. This linear pattern is assumed to be altered as a consequence of either rain or street sweeping.

Equation 2-1

$$X = \frac{a}{b}(1 - \exp^{-bt})$$

Where: X = surface load (g/m^2)

t = time (in weeks)

a = rate of supply (g/m^2/week)

b = removal constant (per week)

The loading rates for the various catchments in the UK study were found to be reasonably consistent with those of the previous US studies.

All of the studies detailed above observed an uneven distribution of sediments across pavement surfaces, with the highest concentrations being observed at the kerbside (Figure 2.14). This is not unexpected, as this location is generally the low-point of the carriageway cross-section and offers some shelter for wind and traffic blown particles.

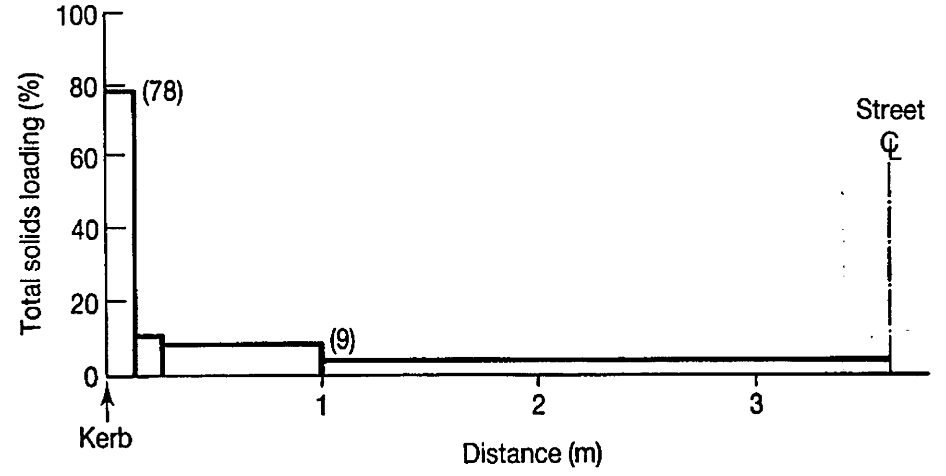


Figure 2.14 - Spatial distribution of surface sediment (Sartor & Boyd, 1972)

2.3.2.5 Domestic and Industrial Inputs

Since one of the principal design criteria for combined sewers is the conveyance of domestic sewage, its role as a contributor to total sediment loadings is significant. The input pattern for domestic sources has been found to vary spatially and temporally. Most studies however, have ignored the temporal variation (diurnally, daily and seasonally) of domestic inputs and give results in the form of average values.

Location	Mean TSS Value (mg/l)
Abu Dhabi (Horan, 1990)	198
Jordan (Horan, 1990)	900
USA (Metcalf and Eddy, 1991)	220
UK (Crabtree et al., 1991)	125

Table 2.5 - National variations in TSS concentrations

The range of concentrations throughout the day is usually large, with solids concentrations falling close to zero during the early hours of the morning. This has important ramifications for deposition studies as it is during this period that flows are also lowest and hence depositional processes can occur. The reduction of solids concentrations at this time therefore reduces the probability of excessive deposition.

The characteristics of domestic solids are more predictable than other types of drainage sediment. Typically domestic solids comprise a mixture of fine faecal matter, gross solids and sanitary refuse. The fine faecal matter (sanitary solids) results from the degradation of large faecal solids (gross solids) as they are transported through the system. Additionally, the broken-down remains of other organic material are also included in this type of input. Studies of combined sewers dry weather flow characteristics have shown that this material has a d_{50} of approximately 34μm (Chebbo et al., 1990) and a volatile content of up to 80% (Verbanck et al., 1994).

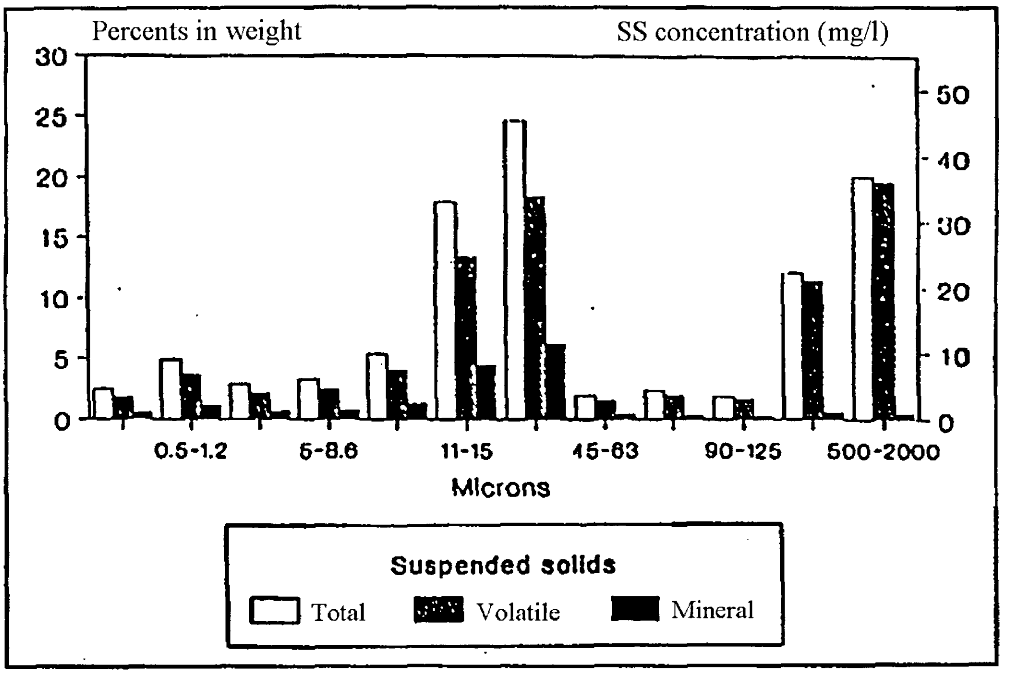


Figure 2.16 - Particle size distribution for sanitary solids from Verbanck et al., 1990

This fine, principally organic material, tends to be of a low specific gravity ($\approx$ 1.0), resulting in mean settling velocities in the range of 1.4 – 1.9 m/h (Verbanck et al., 1994). Alternative studies have revealed settling velocities outwith this range but as a variety of setting column tests exist (Section 3.5.6.2); these figures are not directly comparable. Similarly, differences in the variability of the sediments tested and sampling protocols can result in selective sampling which can strongly skew results. The position of the sampling point within the flow column can have a large bearing on the type of material extracted, as particles with higher settling tendencies are usually more concentrated closer to the bottom. The exact distribution varies with many factors (roughness, turbulence, flow velocity, particle characteristics) making comparisons even with the same sampling protocols at best, difficult.

Little is currently known about the gross solids from which most sanitary solids are derived. Much of the work to date has sought to define gross solids (Jefferies & Ashley, 1994; Friedler & Butler, 1995; Milne et al., 1996). However, as a consequence of the variability of faecal stools, domestic waste and sanitary refuse, no single definition has been reached. A diurnal pattern has been established by Friedler (Friedler et al., 1995) and the volatile content of gross solids has been found to vary from 24 to 81% (Milne et al., 1996). These studies have revealed a flush wave type movement, where the solids pulse through the system (Littlewood & Butler, 2003). On reaching larger diameter pipes with a more substantial base flow, the neutral buoyancy of the particles generally distributes them more randomly throughout the flow column.

Industrial inputs vary greatly in physical, chemical and biological characteristics depending on the industrial process involved. As such, no general conclusions can be drawn and the impact of each industrial input on the drainage catchment performance must be considered individually. Industries using large quantities of water for washing will frequently exhibit high solids loadings in their effluent (e.g. food processing). A widespread industry which has been recorded as having large effects on the operation of sewerage networks as a result of sediments is that of construction. Site activities such as demolition, excavation, groundwater pumping and aggregate storage can provide huge sources of mobile sediment that can be washed or blown into the drainage network. Observations have indicated that construction activity can increase sediment surface washoff loadings by up to 300% (Ashley & Crabtree, 1992). The type of solid washed off in these cases is usually dense (S.G. ≈ 2.2) and principally mineral. Sizes can vary greatly from 100 μm up to full bricks, brieze blocks and wasted concrete.

Construction material such as bricks and concrete can also make their way into the drainage system as a result of the degradation of the conduit fabric itself. Pressure flows, physical impacts and chemical attack slowly break down the conduit wall structure until fragments are released into the flow. Following the decay of the sewer structure, surrounding soil may also slowly migrate into conduits along with groundwater flow. Particle characteristics from all construction activities and surrounding soil are such that problems associated with the deposition of these materials is likely.

2.4 Sewer Sediment Characteristics

Investigations into the types of sediments found in drainage systems have focussed primarily on trying to classify sediments with regard to their physical characteristics. These studies emanated principally from Western Europe, with the UK playing a central role. The most widely used classification system was proposed by Crabtree (Crabtree, 1989). The reason that the classification has become popular is the ease with which it is applied. It is based on a visual inspection and assessment of the location of deposits. Approximate physical characteristics are also given for each classification (Table 2.6). In general, organic content is found to increase with decreasing particle size.

In addition to the classes shown in Table 2.6, there is also type B sediment which is a cemented or agglutinated type A sediment. Although the categorisation provides a simple method of classification, in reality a mixture of any of the above classes may be present at a given location.

Sediment type	Description / where found	Wet density (kg/m^3)	% by granular particle size (mm) minimum – mean - maximum			Organic content (%)
			< 0.063	0.063-2	2-50	
A	Coarse, granular bed material - widespread	1720	1-6-30	3-91-87	3-33-90	7.0
C	Mobile fine grained – found in slack zones in isolation of overlying type A	1170	29-45-73	5-55-71	0	50.0
D	Organic pipe wall slimes & biofilms around flow level	1210	17-32-52	1-62-83	1-6-20	61
E	Fine grained mineral & organic material found in CSO storage tanks	1460	1-22-80	1-69-85	4-9-80	22.0

Table 2.6 - Sewer sediment taxonomy as proposed for UK (from Crabtree, 1989)

In general, the larger more dense sediment deposits tend to be found in locations where velocities are higher. These areas are normally located in parts of the system where pipe gradients are steep and pipe sizes small (i.e. at the head of the system). As sediment is transported through the system, there is a gradual sorting of the sediment with the finest material finally depositing in large interceptor sewers. At present there is a dearth of information on the spatial distribution of sediment deposit characteristics, highlighted by the frequent current use of a single sediment size to represent all areas of a catchment wide sediment transport model.

	Particle size range (mm)	<0.1	0.1-0.2	0.2-2	2-20	>20
Country	Sewer type					
UK	Trunk sewers Interceptor sewers	1-8 1-20	1 1-21	18-41 32-80	41-50 65-20	NR NR
France	Trunk sewers Interceptor sewers	4 4.5	5-10 ←35.5---	38-55 --------→	25-45 ←60-----	8-10 --→

Table 2.7 - Size range percentages for sediment deposits for varying locations (Ashley and Crabtree, 1992)

Table 2.7 shows the percentages of various size ranges present at different locations for the principal European studies. As can be seen, in both the UK and French studies there is a larger percentage of fine material in larger interceptor sewers than measured in the trunk sewers. This suggests that hydraulic sorting does take place within the drainage network.

2.5 Current Sediment Management Practice

In 1995, CIRIA Report 134 (Butler & Clark, 1995) was published which aimed to establish the current sediment management strategies and recommend best practices. CIRIA 134 is therefore used in this section to define traditional sediment management methods. Following a summary of the CIRIA report, alternative sediment management techniques are also reviewed.

At present, responsibility for sediment management in the UK is passed on along with the sediment as it is transported through the catchment from catchment surface to treatment plant. Sediment removal or cleaning is carried out at various stages by local authorities, highway authorities and sewerage undertakers, each under different objectives and legislation (Butler & Clark, 1995).

Figure 2.18 shows the traditional routes of entry, transport and exit for typical drainage catchment sediments. The following sections look at the exit points that play the most significant roles in terms of proactive sediment management.

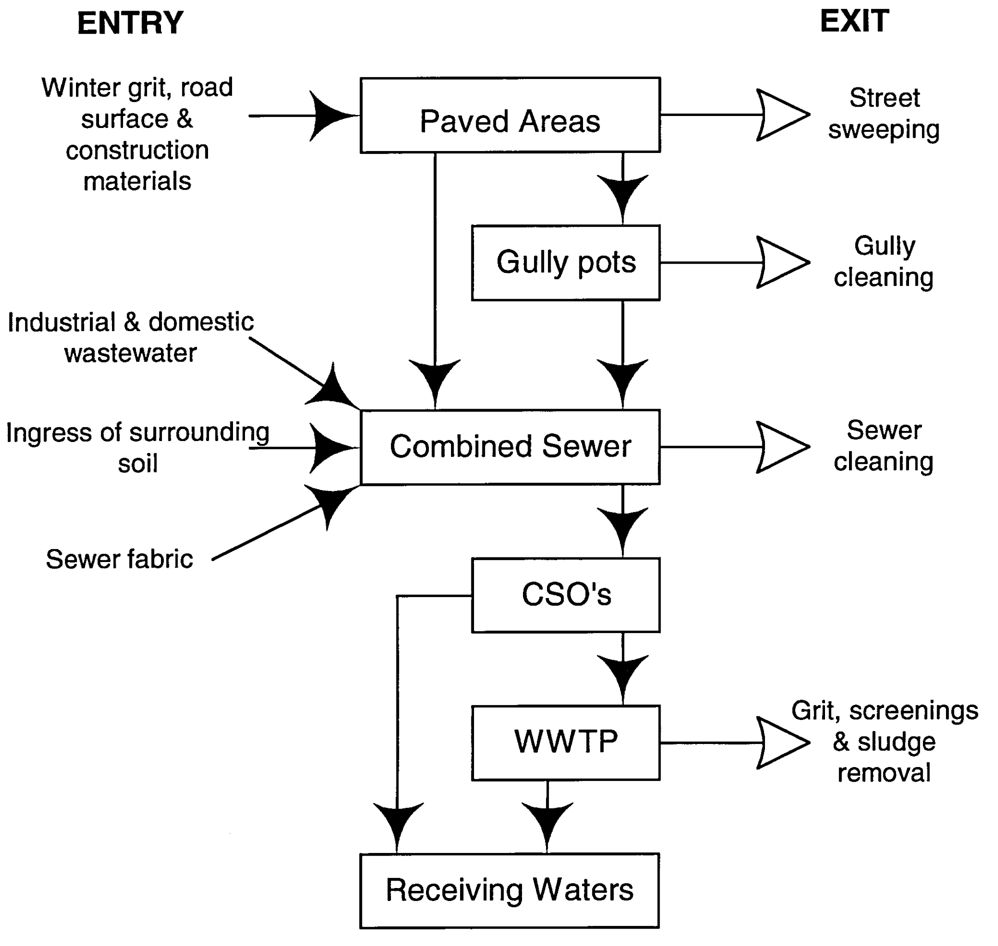


Figure 2.18 - Entry, routing and exit of sediment in traditional drainage systems (modified from Butler & Clark, 1995)

2.5.1 Street & Gully Cleaning

In general terms, any type of substance management is usually most efficiently dealt with at source. However, as previously described, the sources of sediments contributing to drainage systems are extremely diverse. The closest method to source control that is used is the sweeping of streets. However, the impact of street sweeping on reducing the build –up of sediments remains unproven.

Over time, sediments washed off from surface areas will deposit and build up in gully pots. To avoid blockage of the outlet trap and pipe, regular emptying is required. A survey carried out for CIRIA in 1995 suggested that the emptying of gully pots was universally practiced (Butler & Clark, 1995) and suggested cleaning frequencies in the order of:

- more than once a year (78%)
- once per year or less (22%)

The efficiency of gully cleaning practices is not widely documented. The only UK site tests of gully cleaning efficiency were carried out studying 132 gullies in Lambeth and Rotherham. Cleaning efficiencies were found to vary greatly from 20% in one location to 98% in another. The average cleaning efficiency was found to be in the order of 75%. It was also estimated that during the cleaning process over 10% of gully pot sediment is washed into the sewer (Butler & Clark, 1995).

2.5.2 Sewer Cleaning

In locations within a drainage system where sediment will continue to build up to unacceptable levels, or create a potential problem, the deposited sediment should be removed. A number of cleaning techniques are available and are used according to site specific conditions.

The latest generation of sewer cleaning tankers incorporate combined jetting and vacuuming units, silt de-watering and most recently the recycling of sewer flows and silt washings to be used for the jetting process. This significant innovation means that much lower volumes of clean water are used resulting in cheaper and significantly faster operation.

High pressure jets are applied to consolidated sediment deposits to break it into smaller parts. These parts are then abstracted from the sewer as they are dislodged via the suction hose. The abstracted material can now be washed and dewatered within the tanker allowing the tanker to operate for longer without the requirement for tipping the collected material.

As a result of the higher pressures used in recent years, concerns have been raised regarding potential damage to sewer fabric. This is of particular concern in the case of Victorian brick sewers. This has resulted in the publication of the Sewer Jetting Code of Practice (WRc, 1997).

2.5.2.1 Sewer Cleaning Costs

Generalisations regarding cleaning costs are difficult due to the number of factors involved, several of which are summarised below:

- type of material to be removed;
- sewer depth and diameter;
- amount of deposition;
- hydraulic conditions;
- ease of access;
- type of equipment used.

However, data collected during a CIRIA 1991/92 study suggested cleaning costs in the range of £50 to £70 per m^3 of sediment (Butler & Clark, 1993). It should also be noted that current practice when employing any of the sewer cleaning techniques listed above, involves a reactive strategy requiring individual inspection. Inspection costs should therefore also be included for each cleaning method.

2.5.3 Additional Practices

The traditional location for sediment removal is at "end of pipe" treatment facilities. It is therefore preferable that the maximum quantity of solids that get into the sewer system is passed through the system. The use of a minimum, "self-cleansing" velocity in the design of conduits transporting solid material is ubiquitous. Although the practice is applied world-wide, the standards vary from country to country.

Table 2.8 and Table 2.9 show the values of minimum velocity and shear stress in each of the principal design standards. Recent research into sediment transport phenomena has revealed these standards to be inappropriate in most cases and inadequate in the case of larger sewers (Ackers et al., 1996). In addition to the size and shape of sewer sections, other parameters such as sediment characteristics, pipe roughness and sediment loading have been shown to play an important role in the definition of a self cleansing criterion.

Source	Country	Sewer Type	Minimum Velocity (m/s)	Pipe Conditions
ASCE (1970)	USA	Foul	0.6	Full/Half-full
		Storm	0.9	Full/Half-full
BS EN 752 (1997)	UK	Storm	0.75	Full
		Combined	1.0	Full
Abwassertechnishe	Germany	Foul	0.48-2.03	0.3 to Full
Vereinigung ATV		Storm	0.48-2.03	0.1-0.3D
(1998)		Combined	0.48-2.03	0.1-0.3D

Table 2.8 - Minimum velocity criteria

Source	Country	Sewer Type	Minimum Shear stress (N/m^2)	Pipe conditions
Yao (1974)	USA	Storm	3.0-4.0	Full/Half-full
		Foul	1.0-2.0	Full/Half-full
Maguire	UK	Combined	6.2	Full/Half-full
Bischof (1976)	Germany	Combined	2.5	Full/Half-full

Table 2.9- Minimum shear stress criteria

The entire idea of the self-cleansing condition has also recently been brought into question. Laboratory studies have revealed that the presence of a limited sediment bed actually enhances the sediment transport capacity of the flow, and can hence be used to reduce the slope requirement for particular pipes (Nalluri & Ab Ghani, 1993; Ma, 2003).

The response in the UK to this research was the development of a new design code (Ackers et al., 1996). The code utilised predominantly European research and analytical techniques to allow a more detailed assessment of self-cleansing requirements to be made. The principal advances made in the design approach were the allowance of an acceptable deposit depth and the inclusion of sediment characteristics in the analysis. Although the procedure was simplified using design charts and tables, uptake has been slow, as engineers have viewed the procedure as being excessively complex (Arthur et al., 1999).

An alternative to keeping solids moving at all times is the periodic flushing of sewers to temporarily raise local velocities and turbulence in order to re-entrain deposited particles. This technique was used extensively in the design of Roman bathhouses and has existed in various forms since then. The principal methods of flush creation involve either the addition of external water sources to raise flow rates, or the creation of a "dam-break" effect.

In recent years, significant flush studies have been carried out in the USA, Germany and France. The most extensive of these was a long-term study on the pollution reduction effected by a strategy of flushing in Boston, Massachusetts (Pisano et al., 1979). Although encouraging results were obtained from this extensive study, little work was carried out to refine the findings. More recently in Germany and France, studies have examined the possibility of the development of automatic flushing methods.

In Germany, the HYDROSELF system is used to prevent sedimentation in sewers, tunnels and tanks (Grande & Novak, 1995). The system utilises a storage volume of water (either on or off-line) restrained behind a hydraulically operated flap valve. Figure 2.20 shows the arrangment of the HYDROSELF chamber with the hinged flap valve operated by an automatic float system (not shown). In large diameter conduits (> 2000 mm) the storage can be retained within the pipe. For smaller conduits, an off-line storage tank is required.

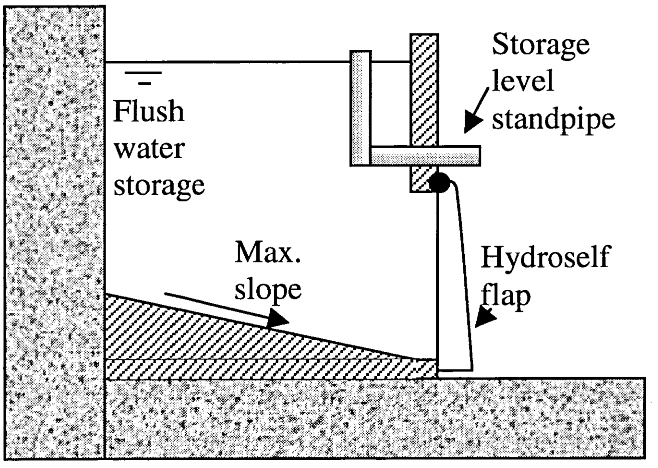


Figure 2.20 - HYDROSELF chamber

Of the 209 European installations, 77 have been used to flush sewers of diameters varying between 250 mm to 4300 mm. In each case the length through which the pipe remained sfre of sediments was recorded as the “flushing length”. The flushing lengths achieved in each of these installations can be seen in Figure 2.22. Under the specifications of the installation, an initial design wave celerity of 2.0 m/s must be achieved in each case.

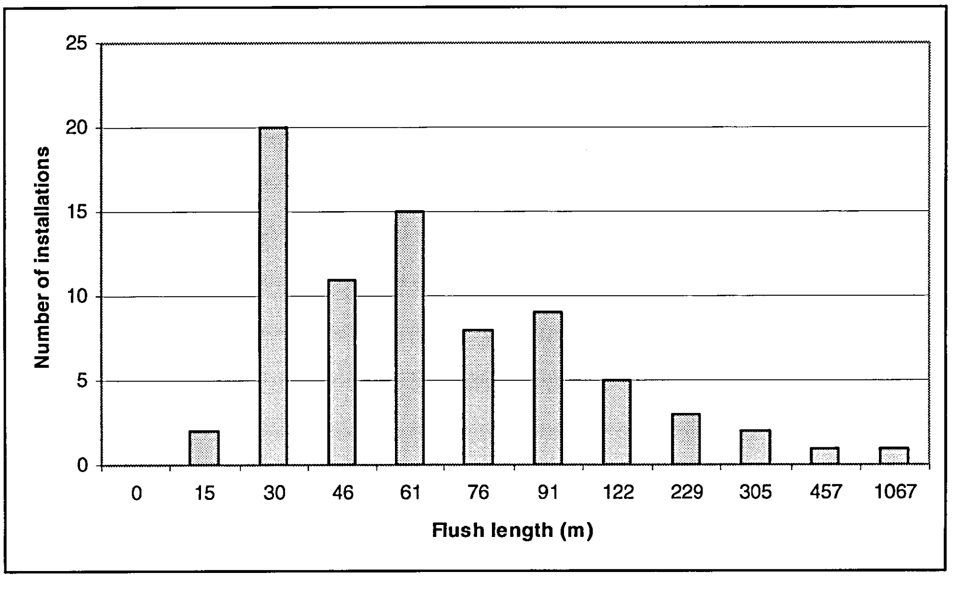


Figure 2.22 - Flushing lengths achieved by various HYDROSELF installations (Grande and Novak, 1995)

The French system varies slightly as it utilises a flow control gate to produce pulses of unsteady flow. The HYDRASS gate (Chebbo et al., 1996) has an off-centre pivot that allows the gate to tip if sufficient hydrostatic pressure exists (Figure 2.24). Once the wave is released, the self-weight and water pressure exerted on the gate reinstate the gate to its original vertical position. The process then repeats. These gates have been installed extensively in Marseille and have shown that frequent flushes are effective at reducing sediment deposition.

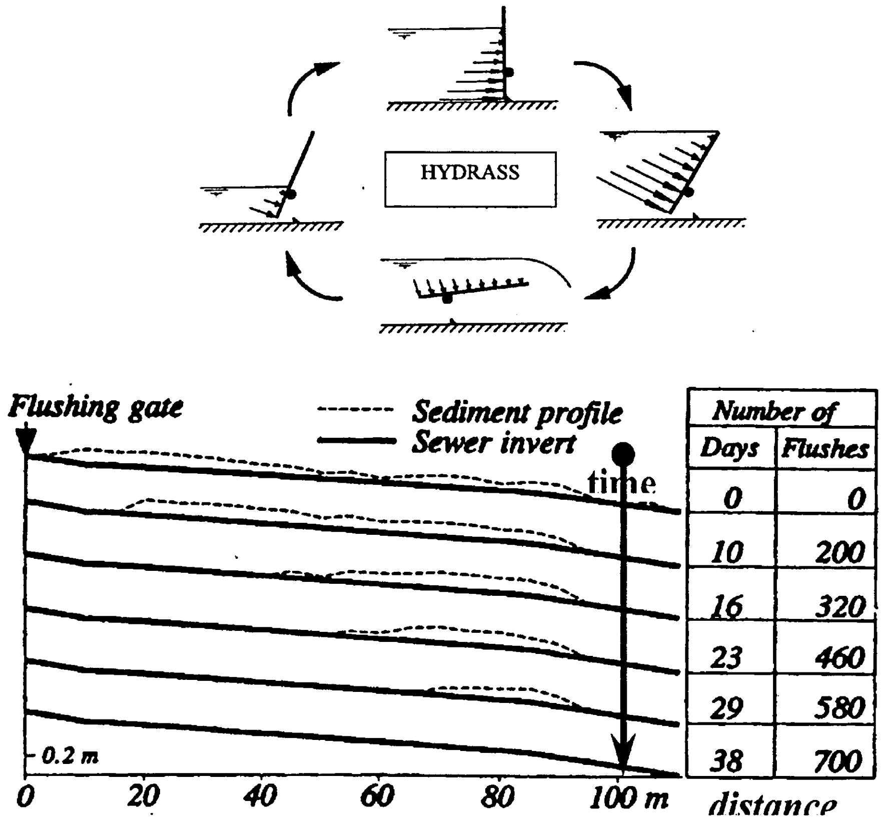


Figure 2.24 - Hydrass gate (Chebbo et al., 1996)

Figure 2.24 illustrates the operation of the HYDRASS gate and shows experimental results from tests carried out in trunk sewer number 13 in Marseille. The profile of sediments located downstream from the gate can be seen to diminish gradually over time, with the pattern of erosion starting at the upstream end and working downstream. In this particular case the initial average sediment depth was 100 mm.

Alternative gate designs are currently being tested which allow the gates themselves to slowly migrate down through the system flushing and physically pushing sediment as they move.

2.5.4 Sediment Management Discussion

Of all the sediment management options detailed above, all but the flushing methods are essentially reactive responses to performance problems highlighted by some clear symptom (e.g. flooding in wet weather). As such, these are often expensive solutions to expensive problems. Methods such as street sweeping and gully emptying tackle some of the problems at source but would be too expensive and labour intensive to employ on a wide but detailed scale. Source control of other sources such as construction material and soil ingress is much more difficult to police and control. There is therefore little that can be done to entirely exclude sediments from drainage systems. Techniques such as the flushing gates can be used in localised problem areas to move the sediment on to an area of higher sediment transport capacity. At present the long term maintenance requirements of the gate's moving parts are not known, although early evidence suggests that this should not be problematic (Laplace, 1999). However, in many cases, improved areas of sediment transport may not exist and it is these areas that an alternative is really required. In these cases it is proposed that sediments should be separated from the flow and stored so as to minimise the risk of re-suspension. The use of sediment traps facilitates this and is discussed further in Section 2.6.

2.6 Sediment Traps

The simplest and most common method of solid/water separation is via settlement processes. Sediment traps utilise these processes to allow undesirable solids to be removed from the transport phase and held in storage. Although sediment traps have been in limited use since the 1890's (Ashley et al., 1995), little research has been carried out into the design or operation of such structures. The technique of in-sewer sediment traps gradually lost favour after the Victorian era due to the labour intensive and unpredictable nature of the maintenance required. However, modern mobile vacuum units and enhanced knowledge of sediment behaviour make the use of sediment traps again viable.

There are at present no formal design guidelines for sediment traps available within the UK water industries (Fraser et al., 1998). Limited design work has been carried out in the UK using modified settling basin theory. However, given the wide variety of flow conditions and the turbulent nature of the flow at a sediment trap inlet, many of the design assumptions of settling basin theory are invalid. New guidelines are currently being prepared in France, with little additional information available from world-wide research centres (USA, Holland, Germany and Australia). Within the UK, the only reference in current best practice guidelines comes from CIRIA Report 134 – Sediment management in urban drainage catchments (Butler & Clark, 1995). The report advises that design principles should follow good hydraulic practice and avoid turbulent conditions. A "long narrow tank" and minimum flow velocity of 0.3 m/s are also advised. These design principles offer good general advice but do not address the difficulties of the designer which are essentially:

- Where should the trap be located?
- What form should the trap take?
- What size will the trap be?
- How frequently will the trap require cleaning and maintenance?

The most detailed studies to date on sediment traps have taken place in France, where long-term studies have shown the potential for reducing sediment management budgets by employing a strategy of trapping sediments (Paitry et al., 1990). Further French studies have also shown that various design modifications can enhance the performance of the trap and make it more selective in the type of material it retains (Dartus, et al., 1990; Paitry et al., 1990; Bertrand-Krajewski et al., 1996; Laplace and Felouzis, 1997; Buxton *et al.*, 2001). This improved precision of understanding and specifying performance highlights the need to initially define the role that should be performed by an ideal sediment trap.

2.6.1 Purpose of sediment trap

The principal purpose of the sediment trap is the retention of sediment particles which would otherwise deposit in a sewer and create hydraulic and pollution problems. Given the range of particles in motion within drainage systems, it is necessary to define which particles cause these problems. Traditionally, large dense particles have been viewed as the main contributors to hydraulic problems, and fine organic particles largely responsible for the polluting potential of bulk deposits. Observations of sediment deposit characteristics have supported this, although fine sediment particles can become associated with granular deposits once a sediment bed is established. Given the large volumes of sediment transported each day in a typical drainage system, it is therefore preferable from a maintenance viewpoint, that only the particles which can deposit downstream are held in the storage volume of a trap. If all sediment particles are retained, the frequency of emptying and cost of disposal of the material will be unacceptable. Invert traps should therefore aim to select only material with high settling velocities and allow low density, polluting material to pass on to treatment facilities. As principally mineral material is retained, degradation and gas production in the trap are significantly reduced.

2.6.2 Previous Sediment Trap Studies

Sediment traps have been in use in drainage systems since the Victorian era. At this time they would have been a necessity due to the large sediment loadings and subsequent high deposition rates of the time. The sewerage system of Dundee would have represented a “state of the art” in many respects at the time of its construction. Its egg-shaped sewers, in addition to the provision of a planned network of sediment traps and flexible flow control gates allowed a level of proactive maintenance rarely seen before or since.

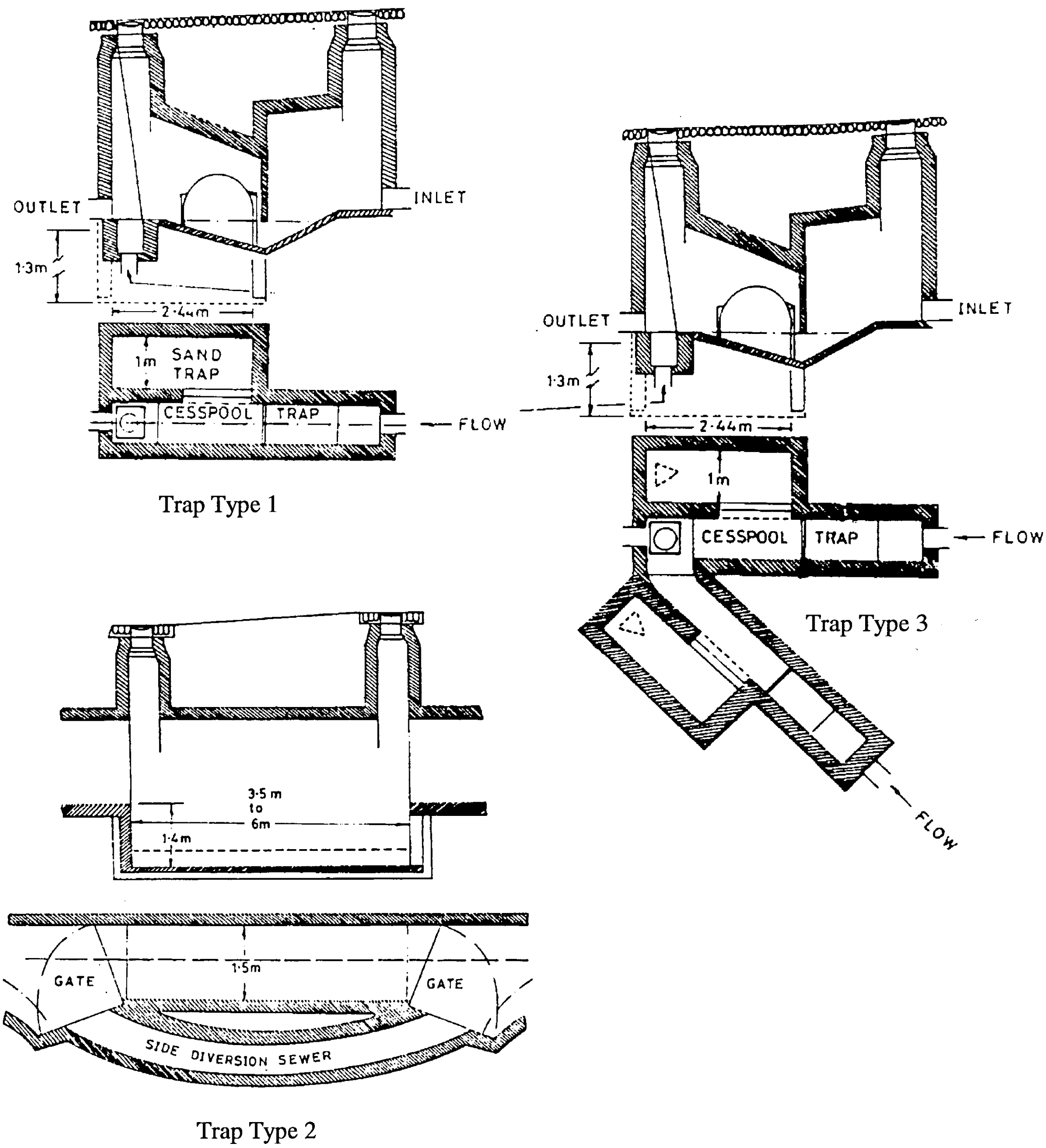


Figure 2.25 - Victorian Trap Types (Dundee Advertiser, 21st January 1884)

Figure 2.25 shows the three trap designs conceived by Dundee's sewerage designer, John Bateman. Trap types one and three are of the same basic design with slight variations in layout. The design acts as a combined "gas and sand trap", with the central vertical lip intercepting the surface of the flow and hence preventing the downstream migration of gases. The sand trap is an off-line storage structure that attempts to encourage sediment to gravitate from the central sump along a transverse gradient into the storage chamber. Little is known about the performance of this type of trap, but as few remain today it can be assumed that they were not found as effective as trap type two. Trap type two is a simpler design with a sudden drop in the invert level forming an on-line storage volume. This type of trap is the largest of the types available and is also the most commonly found trap still in existence in the UK (Fraser et al., 1998). Although in operation for over one hundred and fifty years, no research into the performance of these traps was carried out until the early 1990's. This work was instigated by the UK research programme into sewer sediments and also early sediment trap studies in France.

The early work on the performance of sediment traps focussed on the observation and conceptual modelling of French grit chambers. The French grit chamber design is characterised by a rapid drop in the pipe invert and an associated widening of the flow cross-section (Figure 2.27).

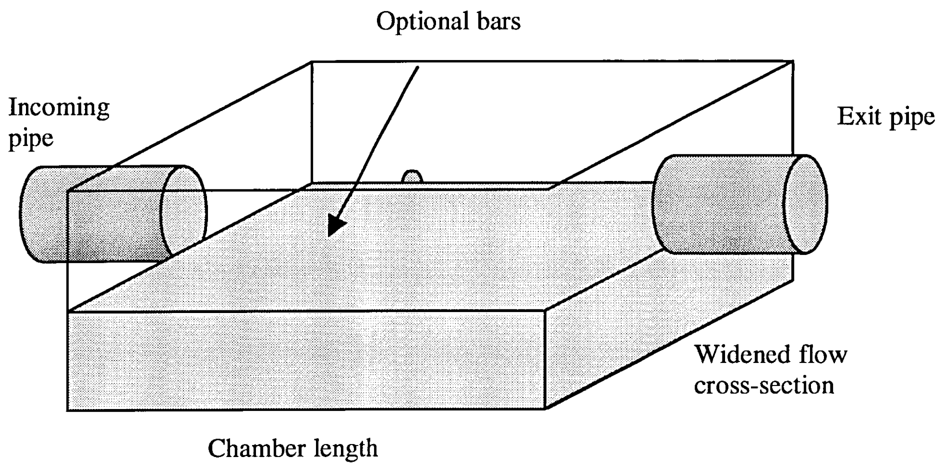


Figure 2.27 - French grit chamber schematic

This results in the slowing of the flow and the settlement of up to 70% of the incoming solids (Dartus & Alquier, 1985). This high rate of settlement was improved further by the insertion of flow obstructions such as metal bars (Figure 2.27). The partial flow obstructions slow velocities further and reduce re-circulations that can reduce deposition rates and cause re-erosion. A large-scale data collection exercise was undertaken by the Water and Sewerage Depatrment of Seine Saint Denis, encompassing 100 grit chambers. The general pattern of filling found in these early studies was that of deposition during storm events and slight erosion during peak dry weather flows and low intensity storms. This pattern went against the general assumption that dry weather solids were the principal source of chamber filling. However, it should be noted that in these studies an area of extensive upstream deposits was observed. Hence, typically, dry weather sediments would be likely to deposit at an upstream location, only to be re-eroded during storms, then intercepted by the grit chamber. It is believed that the observations of sediment erosion during less substantial flows are a result of three dimensional flow effects only present at lower inlet velocities as a result of the oversized chamber dimensions (Dartus et al., 1990). The minimisation of these erosions and prediction of fill rates were the main objectives of these studies. It was also obsereved that the filling pattern was generally asymptotic, levelling off at around 80% of the grit chamber's capacity.

Using a mass balance analysis of the sediment entering the system and the observed efficiencies of eight grit chambers in the Toulouse area, a fill rate model was developed giving resultswith less than a 10% variation from observed figures (Dartus et al., 1990). However, as a mass balance approach was used, an accurate measure of deposition rates is required to apply the model. These data are rarely available, making widescale application difficult. Furthermore the observed efficiencies for each trap were seen to vary, even for chambers of identical dimensions.

Following the results of research programmes linking the bed-load type movement of solids with the type of solids depositied in sewers (Ashley & Crabtree, 1992; Lin, 1993), the role of the French grit chambers was questioned. Studies now suggested that deposition problems could be prevented by locating chambers upstream from areas of deposits (Bachoc, 1992) and targeting bed-load material (Bertrand-Krajewski et al., 1996; Chebbo et al., 1996). Small scale slective traps were tested at two experimental modified grit chambers in Bordeaux in 1993 (Figure 2.29).

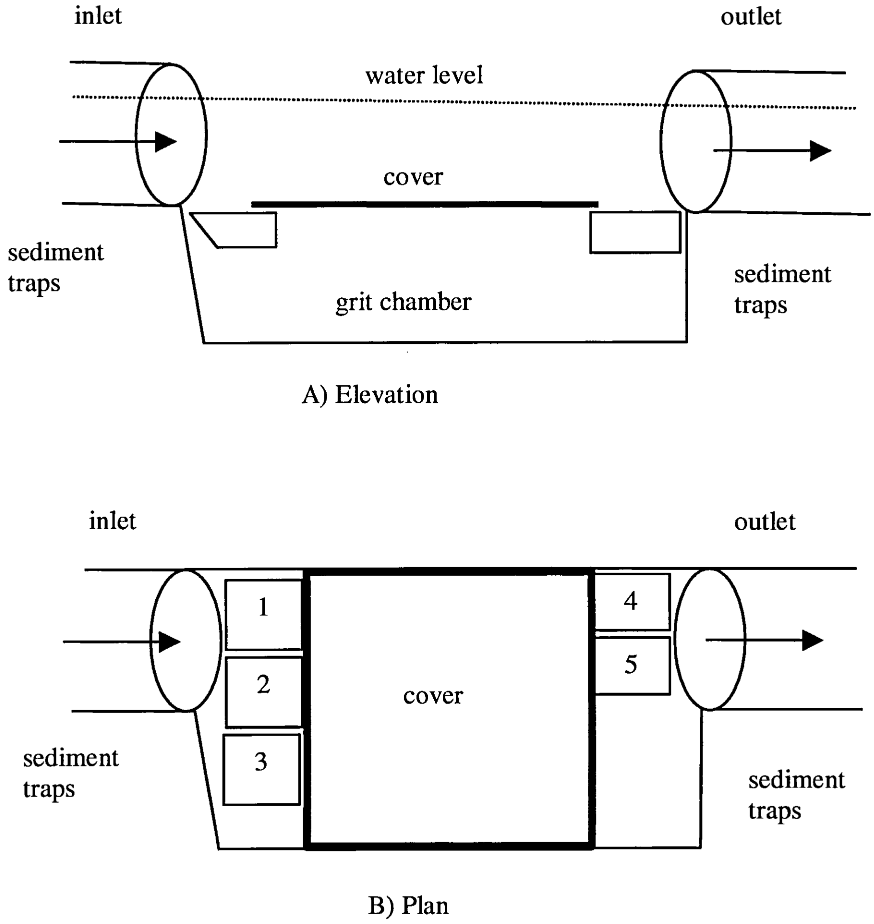


Figure 2.29 - Grit chamber and equipment in Bordeaux

These partial covers were found to reduce the organic content of the material retained to around 20% (grit chamber organic content = 60%), with the mean particle diameter increasing from around 200 to 400 μm (Bertrand-Krajewski et al., 1996).

At this time, a parallel programme of research was also undertaken by the engineer Dominique Laplace in Marseille. His studies tested a selective trap design at full scale using hydraulically powered partial covers over the grit chamber. He hypothesised that in order to intercept only bed material, the slowing down or obstruction of flow was undesirable. Consequently, the widths of the grit chambers were reduced to the diameter of the incoming sewer pipe. By using the partial covers, the storage volume of the chamber could be used without the deposition of fine material and the re-entrainment of material observed at traditional open chambers.

Initial rules for the sizing of these traps were based on storage volume of 0.1 m^3/ha/month. Maintenance recommendations suggested that the traps should be cleaned once per month in dry weather and immediately after significant rainfall. These operational procedures are unlikely to be accepted in the UK as a consequence of differences in climate and government investment strategies. The inflexibility and site specific nature of the Marseille rules were later identified as limitations and further studies are continuing to examine methods of trap filling prediction, the detailed processes and the design features required.

These further studies included work in Bordeaux, examining the effectiveness of traps in reducing pipe deposits. The fill rates of the traps were observed whilst simultaneous measurements of pipe deposit levels, rainfall and sediment transport rate allowed correlations between these factors to be attempted. The operation of the trap was seen to significantly reduce downstream pipe deposition over a period of 200 days (Figure 2.31).

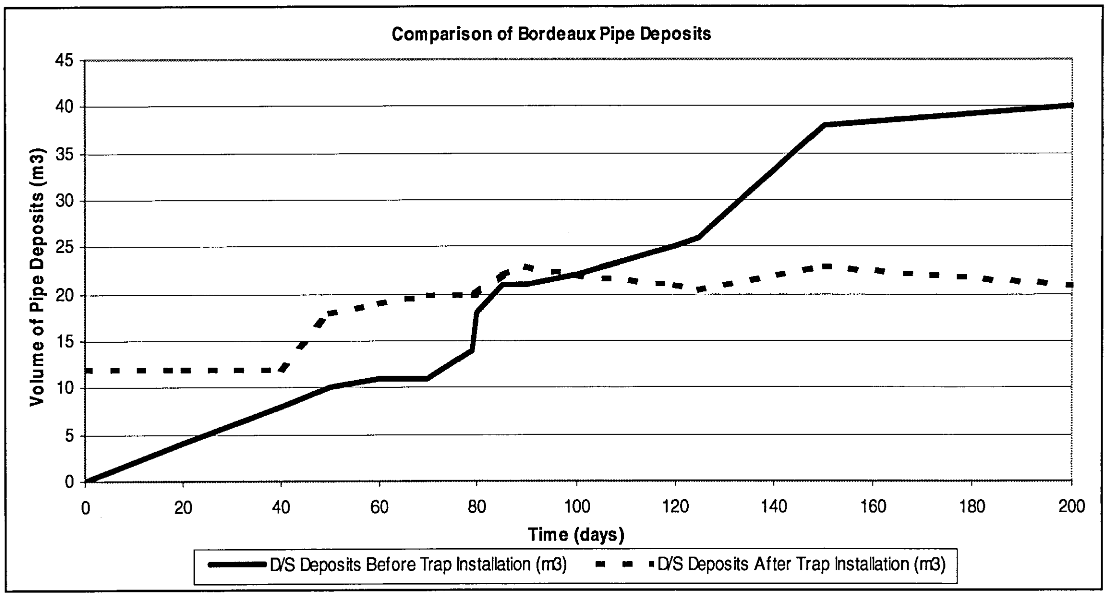


Figure 2.31 - Accumulation of deposits in the downstream pipe (from Laplace *et al.*, 1999)

2.6.2.1 Sediment Trap CFD Studies

The most recent stage of these investigations was the development of a computational fluid dynamics (CFD) simulation of sediment trap behaviour. CFD has historically been used principally in mechanical engineering for the design of components working within a fluid environment. Extremely detailed information of fluid pressures, velocities, friction and turbulence are used to develop an overall pattern of fluid behaviour. Its application to the area of sediment transport in drainage systems is comparatively new, with the first work being carried out on the design of individual structures such as settling basins and overflow chambers (Stovin & Saul, 1994). Subsequently little work has been carried out on the use of CFD in sewers to predict sediment movement. Although detailed, the CFD analysis can take a large amount of computational time, making long-term simulation impossible. Consequently, for realistic simulation times, a simplification must be used.

Attempts were made within the French research group to utilise a CFD density current model to represent the behaviour of material moving near the pipe invert or bed. A lower fluid layer is implemented in the model characterised by a higher density and viscosity than the principal fluid in transport. Such assumptions are not valid in the realms of sediment transport as the types of material and their motion vary greatly to that of a smooth fluid movement with no mixing. The density current model was however validated using historical experimental results (Bonnecaze et al., 1993) and used to provide an input of sediment to a simulated sediment trap (Schmitt et al., 1998). The model was applied to a variety of French trap configurations, with variations in slot width, location and the relative level of the trap covers. In each case, the fill patterns, re-circulations and erosion were observed until the trap would no longer accept any of the dense undercurrent. At this point the trap was defined as full. Following these tests, the initial empirically supported premise of the most efficient design requiring a central slot position, slot width of 30 cm and level trap covers was confirmed. However as a consequence of the approach used to simulate the sediment inputs, analysis of the selectivity of the trap was not possible. This is the principal failing of the density current CFD investigation.

2.6.2.2 Buxton (2003)

As part of a parallel programme of research into the applicability and performance of sediment traps, a series of laboratory and computational experiments were undertaken at the University of Sheffield (Buxton, 2003). The principal aim of Buxton's research project was to develop a computationally based method for the prediction of invert trap sediment retention performance. Open and partially covered trap configurations were tested and compared.

Computation Fluid Dynamics (CFD) software (FLUENT) was used to simulate the sediment retention performance of a range of invert trap configurations over a range of hydraulic conditions and sediment characteristics. Sediment transport and deposition were simulated using stochastic particle tracking. For the purposes of CFD model verification, detailed laboratory experimentation was carried out using Particle Image Velocimetry (PIV) to obtain highly detailed flow velocity data.

A Reynolds stress turbulence model using the Quadratic Pressure-Strain model was found to give the best overall predictions, successfully recreating the 3-D flow structures within the channel and the invert trap.

Particle tracking was used within the CFD model to predict sediment retention performance for the cases investigated during the laboratory studies. Parametric analysis was carried out to assess the sensitivity of the predictions to a range of parameters that were used to calibrate the model. After initial calibration, the resulting model displayed an excellent correlation with laboratory data.

The representation of a full-scale trap model was undertaken using the Forfar 900mm trunk sewer trap as a test case. Although the outcomes of this work were inconclusive as a result of the field data collection difficulties described in Section 3, some of the more unexpected behaviours of the Forfar trap were represented within the CFD model. Most notably, the observation of increased particle trapping at low values of applied shear when using the partially covered trap were replicated in the model. At the outset of the CFD modelling work, it was not believed that this behaviour could be included. Figure 2.33 (below) shows a non-dimensional plot of trap retention ratio versus sedimentation parameter for both a fully open and 0.1m slot configuration (partially covered). It can be seen that in general the open trap is much less selective than the partially covered configuration, with a greater retention ratio predicted for the majority of values of sedimentation parameter. However, it can clearly be seen that at the lowest sedimentation parameters used, the slotted trap actually retains more material. This behaviour was observed in the field with increased trapping of fine material observed using the slotted trap at low shear stresses.

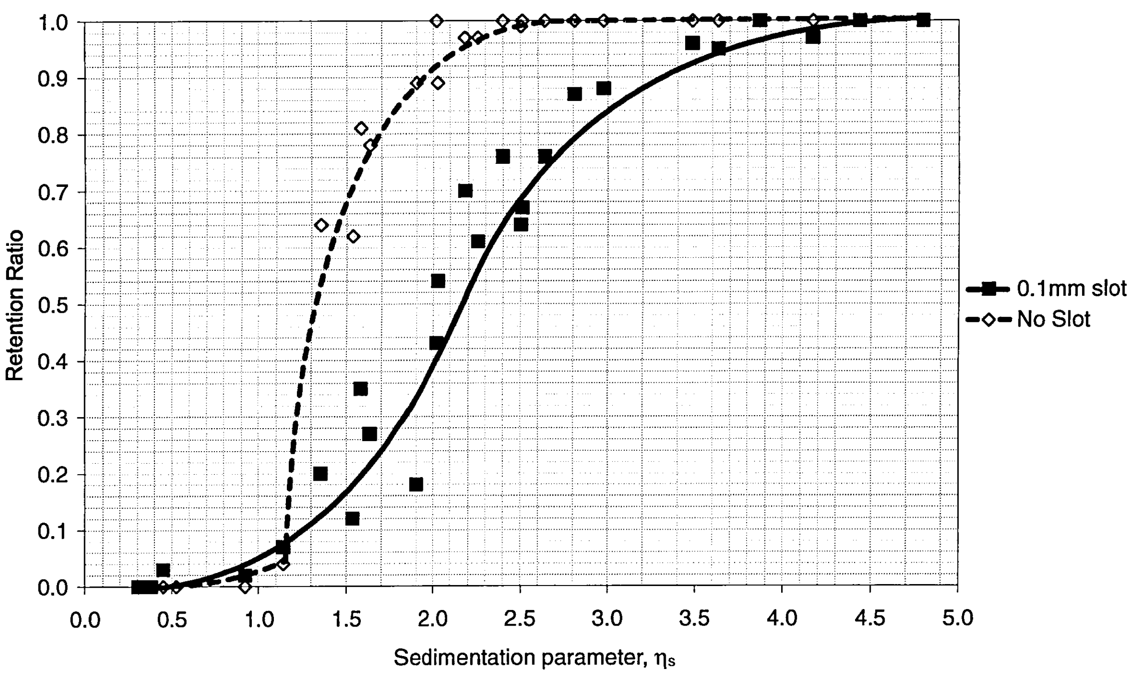


Figure 2.33 - Non-dimensional CFD simulated retention ratios for the full-scale open and 0.1 m slotted trap configurations (from Buxton 2002)

As the focus of the Buxton study was more concerned with the development of a methodology for future CFD studies, certain areas of trap filling behaviour could not be fully investigated. One significant area is that of the changing behaviour of the sediment trap during filling. This is particularly applicable to the case of open traps as retention rates have been observed in the field to reduce rapidly as the trap fills. The changing behaviour of the partially covered traps is not fully known, the experiments undertaken in Forfar proved inconclusive as a result of the hydraulic conditions in the vicinity of the trap.

2.7 Sediment Transport Theory

The methods of predicting the movement, deposition and erosion of sewer sediments are central to devising a sediment management strategy. The following can only be accurately predicted using appropriate analytical techniques:

- estimates of the quantities of sediment arriving at the trap;
- type of material arriving at trap;
- indication of likely locations of sediment deposition;
- indication of volumes of sediment deposition.

A description and evaluation of the appropriate techniques can be found in sections 2.7.1 to 2.7.3.5. Research into sediment transport has historically been centred on the field of alluvial open channels and coastal processes. The application and development of these techniques to an urban drainage environment is a comparatively recent occurrence (late 1980's). However, differences have been seen to exist in the realm of urban drainage that may well limit the application of alluvial-based methods (Verbanck et al. 1994; Berlamont & Torfs, 1996):

- sewer solids are complex mixtures of cohesive and granular materials, often in stratified layers;
- temporal and spatially variable effects are more significant in the case of urban drainage;
- turbulence and boundary layer effects are more complex and have greater effect;
- sediment supply is limited in drainage systems hence, ambient sediment transport loads are often less than the transport capacity.

2.7.1 Types of Sediment Motion

Traditionally, the movement of sediment in a hydraulic flow field has been divided into three transport types:

- wash-load – Fine, low density mobile material. Moves at same velocity as flow. Material rarely (if ever) comes into contact with the pipe invert or sediment bed.
- suspended-load – Particles of varying characteristics. Material kept suspended in the flow by turbulent eddies imparting upward forces. Moves at velocity slightly lower than ambient flows. Occasional impacts with the invert or bed, and mixing with any bed-load material.
- bed-load – Higher density (usually mineral) material moving near the pipe invert at a velocity which can be substantially less then the ambient flow velocity. Saltating motion combines rolling, sliding and bouncing movements.

These modes of transport are shown diagrammatically in Figure 2.35.

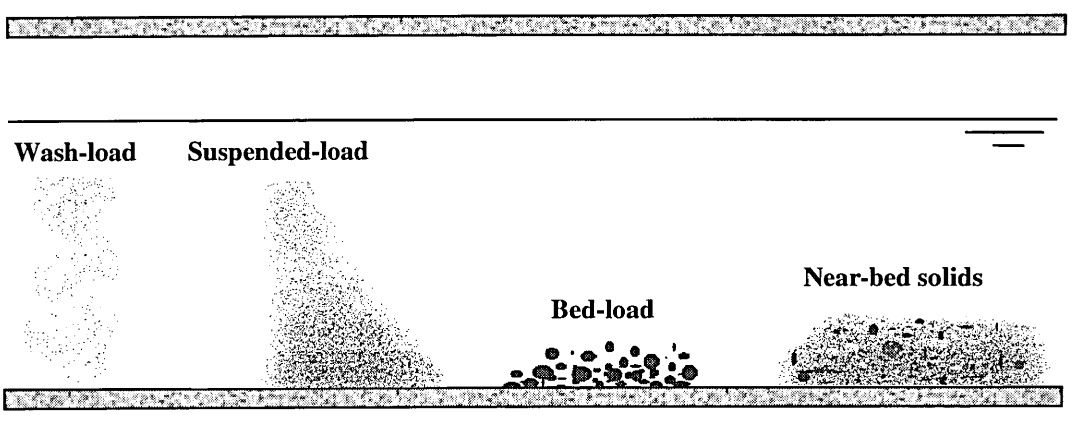


Figure 2.35- Modes of sediment transport

Although these modes of sediment transport and material types are viewed as being distinct, under various hydraulic conditions the material will mix and will interchange from one transport mode to another. The points at which a particle will transfer from one mode to another depends upon the balance of the settling velocity of the particle (w_s) and the energy of the flow. A classification system for alluvial channels based on this balance of forces (Raudkivi, 1990) was adapted by Ashley and Verbanck (1996):

$bedload \Rightarrow 5 < \eta < 15$ **Equation 2-3**

$suspension \Rightarrow \eta \leq 3$ **Equation 2-5**

This classification is based upon the settling parameter, η (Equation 2-7).

$$\eta = \frac{w_s}{\kappa u_*}$$ **Equation 2-7**

where: w_s = particle settling velocity (m/s)

κ = von Karman's constant

u_* = shear velocity (m/s)

In recent years (mid 1990's onwards) the additional mode of transport of near bed solids has been observed at sites in the UK, Germany and France. The common factors in all of the observations have been that the material moves near the bed or pipe invert, but does not exhibit the classical mineral characteristics of traditional bed load material. Samples of solids were extracted from an area directly above the sediment bed using small diameter (≈ 10 mm) sampling tubes (Wöhrle & Brombach, 1991; Ashley et al., 1994; Verbanck, 1995; Ristenpart et al., 1995; Arthur, 1996), indicating a highly concentrated zone in this area. In addition to this, visual observations and detailed measurements have been taken of a similar phenomenon in French sewers (Chebbo et al., 1998).

The various investigations have made different assumptions regarding the way this concentrated layer behaves. Verbanck has worked on the assumption that the layer is an extension of a suspended solids concentration profile and has achieved methods of predicting concentrations in the near bed area (Verbanck, 2000). The procedure projects an assumed concentration profile based on a single "known" or calculated reference concentration at a known height above the pipe invert. However, the accuracy of profiling methods is often outweighed by the inaccuracy of determining a reference concentration. A relationship is provided in the procedure of Verbanck to determine this concentration, but wide-scale testing of the entire method has been limited to date.

The observations in Dundee (Ashley et al., 1994; Arthur, 1996) led to the assumption that the near bed material is of a more variable nature, containing gross solids, fine material, toilet paper and sanitary products. The exact motion of this material has not been established but it is believed to be more of a random, bouncing motion as a consequence of the different particle types. As a consequence of this variability, an empirical model to predict the rate of near bed solids movement was developed for the catchment of Dundee (Arthur, 1996).

The French investigations have concentrated on achieving a high detail of data collection in a spatially restricted area. Hence, although a great deal of information is available, general conclusions are difficult to make. However, observations have suggested that this material is in fact usually stationary and acts as a temporary storage of fine sediments that do not attach themselves to the "semi-permanent" deposits (Chebbo et al., 1998).

These varying observations have led to a range of terms being used to describe the phenomenon including:

- heavy fluid layer
- fluid mud
- fluid sediment
- dense undercurrent
- organic near-bed fluid
- near-bed solids
- la crème

A concerted effort by all of the various research teams investigating near bed solids has allowed some conclusions to be drawn regarding the nature of the material observed (Chebbo et al., 2002). There are three typical approaches:

- sampling throughout the depth of the water column using (typically) small-bore tubes connected to vacuum pumps (Wöhrle and Brombach 1991; Coghlan 1995; Verbanck, 1995; Ristenpart 1995; Arthur 1996; Ahyerre 1999);

- using conventional bedload traps in sewer inverts (Lin, 1993; Coghlan 1995; Arthur 1996);
- direct in-situ visual observation using transparent sewers, boxes and an endoscope respectively (Arthur, 1996; Ahyerre 1999; Oms *et al.*, 2002).

Analysis of the types of material moving in this way has indicated that this sediment is predominantly organic in nature with a high polluting potential. This has significant impacts on the use of traps to control sediment movement, as the traps typically target mineral material moving near the bed. Should this classical bed load be replaced with an organic material moving in a similar manner, the trap will intercept the organic material, which should ideally be passed on to treatment facilities. Further studies and collaboration are required for this mode of sediment transport to be understood, as at present it is unclear if the various studies are observing the same type of material and motion. Current research indicates that the common factor linking the occurrence of near bed solids is the presence of low ambient flow velocities. Consequently, the proximity of a sediment trap to an area where there are near bed solids should be avoided. The extensive methods used to determine sediment transport rates in each of the modes described above, are detailed in Section 2.7.3.

2.7.2 Initiation of Motion

The initiation of motion (also termed incipient motion) is the term used to describe the point at which granular material will just start to move along the pipe invert or sediment bed. In laboratory tests, the initiation of motion is found by gradually increasing the shear stress (τ_o), until a point is reached at which limited particle movements can be observed at a number of small areas over the bed. The shear stress at this point is termed the critical shear stress (τ_c). Any further increases in the shear stress above this critical value will generate a widespread sediment motion (initially as bed load). The initiation of motion is therefore often used to determine the point at which sediment erosion will take place.

2.7.2.1 Shields Criterion

The most widely accepted method of estimating this threshold of movement (and hence the critical shear stress) is through the use of Shields' erosion criterion (1936). Developed using alluvial sediments, Shields reasoned that particle entrainment must be a function of the Reynolds number at the sediment grain (as it is the level of turbulence which provides the upward forces to initiate particle motion) and the settling characteristics of the sediment. Using these assumptions he developed two relationships to represent these two important factors.

$$Re_* = \frac{u_* d}{\upsilon}$$ **Equation 2-9**

where: Re_* = Reynolds number at the grain
u_* = shear velocity (m/s)
d = particle diameter (m)
ν = kinematic viscosity of fluid (m^2/s)

$$Frd^2 = \frac{\tau_c}{\rho g d(S_s - 1)} = \frac{u_*}{(S_s - 1) g d}$$ **Equation 2-11**

where: Frd^2 = Shields' entrainment function
τ_c = critical shear stress (N/m^2)
ρ = density of fluid (kg/m^3)
d = particle diameter (m)
S_s = particle specific gravity
u_* = shear velocity (m/s)

Shields' entrainment function (Equation 2-11) represents a ratio of shear to gravity forces and takes a form used by many of the following sediment erosion studies (Ackers, 1984; Ackers, 1991; Perrusquía, 1991; May, 1993). These relationships have been tested on a large number of data sets since their initial development, with the results used to plot a "critical" line representing the threshold of motion for granular sediments of varying sizes. The resulting Shields diagram (Figure 2.37) can therefore be used to determine the critical condition for a particular sediment particle, or allow practitioners to determine if erosion takes place under a given hydraulic condition. A point plotted on the diagram above the critical line indicates particle erosion, with any points falling below the line indicating sediments remaining at rest.

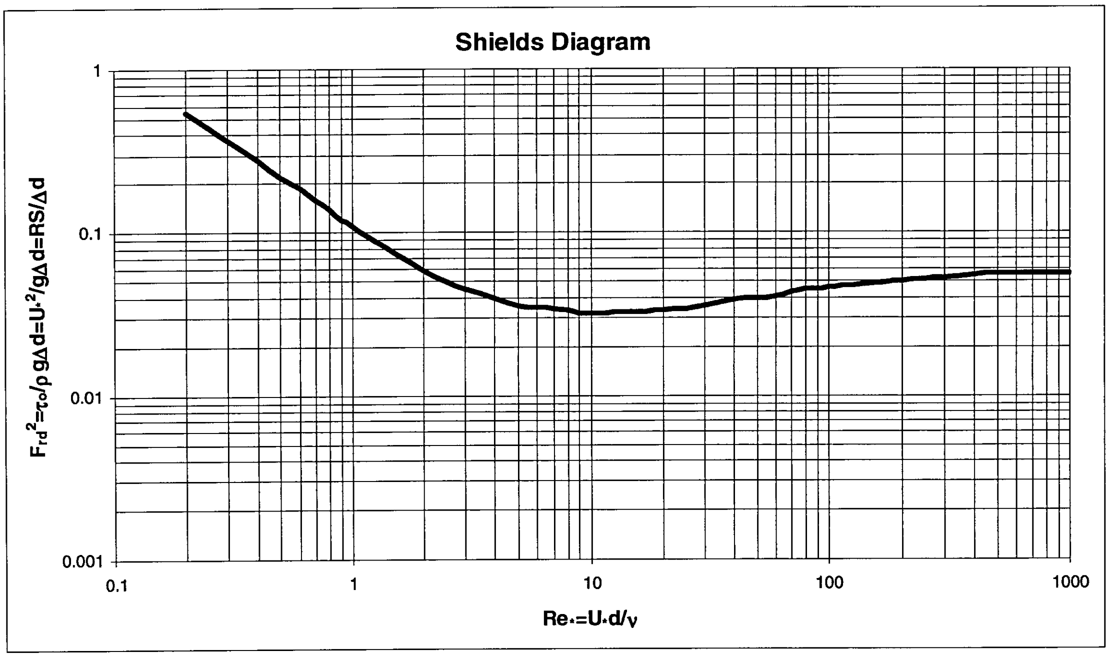


Figure 2.37 - Shields diagram

Although widely used and tested, the applicability of Shields' Criterion to deposits exhibiting cohesive characteristics or mixed particle sizes has been called into question (Skipworth, 1996; Wotherspoon, 1994). Consequently, a limited number of studies have been initiated to examine the implications of these phenomena.

2.7.2.2 Cohesive Analyses

European laboratory studies since the mid 1980's have revealed that that the shear stresses required to initiate the shear failure of sewer sediment samples in a vane test are significantly greater than those required for purely granular samples of comparable particle diameter (e.g. Laplace et al. 1990). Shear vane tests carried out using samples extracted from sites in Dundee, Marseille and Paris have indicated shear strengths of between 2 to 20 kN/m^2. These findings have been supported by more detailed rheometrical tests carried out at University College, Swansea, where yield strengths of more than 2.5 kN/m^2 were recorded (Williams and Crabtree, 1989).

Initially this strength was thought to derive principally from classical cohesive behaviour (electrostatic attraction). However, as a consequence of the diversity of material which compose a sewer sediment bed, the additional strength is more likely the result of a number of combined effects (consolidation, cohesion, agglutination and chemical cementing)

Attempts to measure the behaviour of a sediment bed to varying flow and sediment loadings (Stotz & Krauth, 1986; Nalluri & Alvarez, 1992; Wotherspoon, 1994; Torfs, 1995 and Skipworth & Saul 1995) have produced a number of outcomes. It is clear from these tests that sewer sediments exhibit cohesive like properties which produce yield strengths several orders of magnitude greater than typically applied in-situ shear stresses. This clearly results in the potential for a deposition problem. These cohesive effects have been found to reform in failed samples very quickly (i.e. hours) and have been found to vary with bulk density and moisture content (Wotherspoon, 1994). This dependency upon key variables has led to the development of yield stress models which may be used to predict the total depth of erosion for an applied shear stress. The widespread use of such relationships has, as yet, not taken place. Consequently, a comparison of methods and selection of the approach most suitable to sewer sediment material cannot be carried out.

The principally laboratory based and estuarine experiments carried out to date have revealed a marked increase in bed yield strength with increasing depth (Figure 2.39). This has led many researchers to the assumption that consolidation processes play an important role in the development of bed strength.

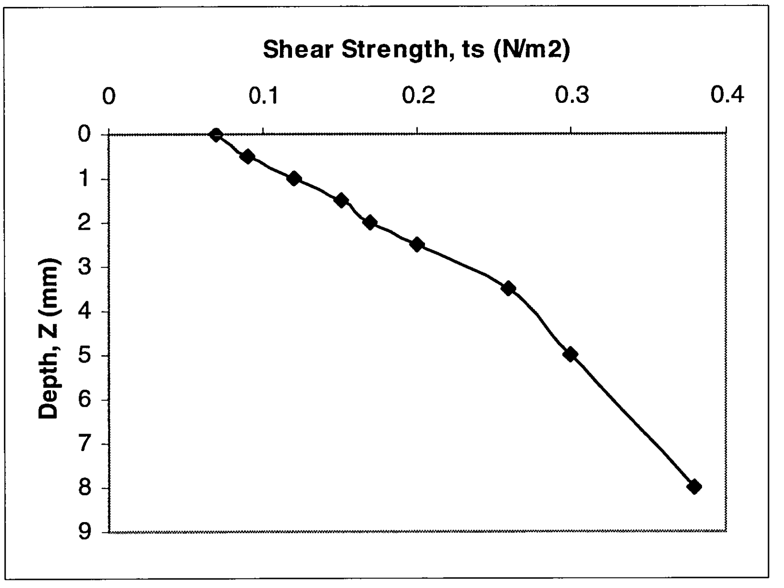


Figure 2.39 - Increase of shear strength with bed depth after Parchure & Mehta (1985)

The formation of a deposited bed requires the processes of settling, deposition and consolidation (Been & Sills, 1981). The initial layer of deposit will comprise the material most readily settled. Consequently, a higher particle density may exist at the base of a deposit at the outset of formation. As further material settles on top of the initial deposit, its self-weight will cause the gradual consolidation of the bed, resulting in a density profile. The rate at which this density increases, varies with the rate of deposition and the type of material, but has been seen to follow an asymptotic profile (Owen, 1970 and Delo, 1991).

Few of the approaches used to determine the density profile of sediment beds have been applied to samples of sewer sediments. The approach of Mehta and Partheniades (1982) has been modified and used to attempt to determine depths of erosion for sewer sediments in the Dundee catchment by Wotherspoon (1994).

A diagrammatic representation of the Wotherspoon model is shown below (Figure 2.41).

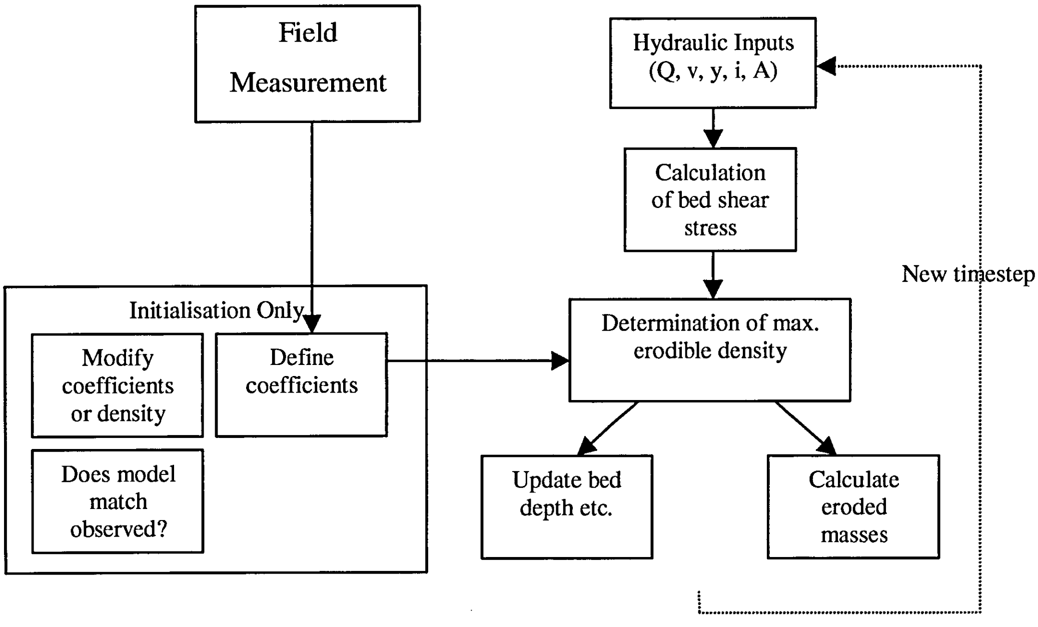


Figure 2.41 - Wotherspoon model flow diagram after Wotherspoon (1994)

In the above schematic, the important variables of yield strength, maximum erodible density and eroded bed depth are given by:

$$yield\_strength = \tau_y = \exp^{18.3865} m^{-3.1682}$$ **Equation 2-13**

Where m = moisture content of sediment

$$\max\_erodible\_bed\_density = \rho_e = \frac{SG\rho_w + e\rho_w}{1+e}$$ **Equation 2-15**

Where SG = sediment specific gravity
e = m SG
ρ_w = density of water (kg/m^3)

$$erodible\_depth = H_e = H_o - \left[H_o \left(\frac{\rho_e}{\varsigma \bar{\rho}_o} \right)^{-\frac{1}{\xi}} \right]$$ **Equation 2-17**

Where H_e = erodible depth
H_o = initial average bed depth
$\bar{\rho}_o$ = average initial bed density

ς and ξ are dimensionless coefficients

2.7.3 Sediment Transport Prediction Methods

The area of sediment transport studies is extensive. A large number of researchers has, over the years, developed a wide array of approaches towards the prediction of sediment transport rates, loads and capacities. As a result of the limited success of the various studies, most methods combine deterministic, empirical and/or statistical techniques. The following sections provide a review of a small sample of the methods available. An assessment was undertaken as part of this study to determine the most suitable method of representing sediment inputs to various sediment management models.

2.7.3.1 Wash-load

The wash-load normally comprises small low-density particles, constantly held in suspension. Consequently, in analytical and modelling terms it is rarely differentiated from the suspended load and is calculated as part of the suspended material.

2.7.3.2 Suspended-load Relationships

Previous studies have shown the significance of the suspended mode in the transfer of solids masses. Ashley et al. (1994) estimated that a total of between 80% to 90% of the total solids transported are conveyed as suspended load. This is as a result of the greater volume conveyed in this way rather than higher concentrations.

A number of suspended solids studies have revealed a concentration profile to be associated with the transport mode (Chebbo, 1992; Ristenpart, 1995; Verbanck 1995). The collection of samples in each of these studies used small bore suction tubes to extract small volumes of the flow at varying heights. Attempts were made in each case to ensure that the velocity of extraction closely matched the ambient flow velocity.

The existence of the profile derives from the variations of flow velocity and turbulence throughout the flow column. Consequently, rules previously used to describe the distribution of turbulent properties (i.e. diffusion) have been adapted to describe the vertical variation of sediment concentrations. The most commonly used form of diffusion based relationships used for sediment transport in streams and conduits are based on the well known Rouse solution (Rouse, 1937). The application of sediment profiling is discussed further in Section 2.7.3.4. A common difficulty with each of the available profiling methods is the selection and estimation of an appropriate reference sediment concentration.

The particles in this suspension profile have been shown to vary in characteristics but are believed to derive principally from sanitary sources (80% organic), with a mean particle diameter of approximately 40 μm.

Few laboratory studies have focussed on the suspended mode of sediment transport as a result of difficulties in replicating, observing, measuring and controlling the behaviour of the fine sediment particles involved. Limited studies in the UK have used the optical measurement of suspended concentrations in studies into the erosion of cohesive material (Skipworth, 1996; Tait et al., 1998). Both of these studies (conducted at Sheffield University) concluded that the Rouse equation was not suitable for predicting the observed concentration profile throughout the entire depth of flow, when based upon a single reference concentration. For the application of a two layer sediment profile model, see Section 2.7.3.4.

2.7.3.3 Bed-load Relationships

For classical bed-load motion to exist, gravitational forces must dominate the vertical motion of particles. The levels of turbulence therefore play a less important role for bed-load motion. Field studies of bed-load sediment transport in sewers have used sediment traps installed on the invert of conduits to determine the characteristics of material moving near the bed. The characteristics of this material were found to be linked closely with the velocities of the incoming pipe (Lin et al., 1993, Arthur, 1996; Fraser et al., 2001). It has been observed that higher velocities and shear stresses result in principally dense granular material moving near the bed in a bed-load manner. Conversely, low velocities have been found to lead to low density, organic matter retained in bed-load traps.

The combination of the lower significance of random turbulent effects and the easier handling and measurement of the sediments involved, results in a large number of previous bed-load investigations. These involve both field (river and sewer) and laboratory studies typically correlating the hydraulic characteristics near the pipe invert or deposited sediment bed with the concentrations of bed-load transport or the rates of transport intercepted by sediment traps. A recent relationship used in this study is that of Perrusquia and Nalluri (1995). This relationship was tested for its applicability to real sewer sediments shortly after its laboratory development specifically for pipe sediments.

$$\Phi_b = 1.49\times10^{-3}.\Theta_g^{2.2}.D_*.Z_{gr}^{-1.11}.\left(\frac{W_b}{y}\right)^{0.78}$$ **Equation 2-19**

Where: Φb = transport parameter (Einstein, 1950)
Θg = grain mobility no. (Einstein, 1950)
y = flow depth
D_* = Dimensionless particle size
Z_{gr} = relative grain size
W_b = Breadth of sediment bed

The differences between the material in both natural systems and sewers are much reduced when considering only bed-load. This results in the increased application of well tested alluvial and river methods to an urban drainage environment.

2.7.3.4 Near Bed Solids Studies

Sediment transport studies in various European sewers during the mid 1990's observed a mode of sediment transport not currently covered by the traditional definitions of wash-load, suspended-load and bed-load transport (Ashley et al.1994; Verbanck, 1995; Ristenpart, 1995).

It was observed that under certain flow conditions, a dense layer of sediment was seen to exist just above the deposited bed or pipe invert. Measurements of the sediment concentrations and characteristics revealed this layer to be very different from traditional bed-load material. Concentrations of material were found to be in a range up to 10 times the order of magnitude greater than those typically found in traditional bed-load conditions. The material was also found to be more organic and more variable than bed-load samples (Arthur, 1996). As a consequence of this variability and the number of researchers observing this phenomenon, a range of terms have been used to describe the transport mode (see Section 2.7.1)

Of the various investigations, two principal approaches have been used to conceptionalise and represent the behaviour of near-bed material.

1. Consideration of the material as a suspension of fine discrete material governed by traditional advection and dispersion laws.
2. Consideration of the material as a mixture of a range of particle sizes and types (including gross and sanitary solids), using field observation to predict rates of transport.

These two approaches have produced two subsequent leading methods for predicting near bed solids transport. These approaches are discussed further in the following sections.

2.7.3.4.1 Verbanck (2001)

Verbanck proposes a two-layer suspension model using traditional turbulent diffusion theory to describe a concentration profile in a 2 dimensional plane. The variation in sediment concentration is determined using the theoretical eddy diffusivity (ε_s), based on the hydraulic inputs of bed shear (u_*) and depth (y).

$$\frac{C_y}{C_{a*}} = \left(\frac{y}{a*}\right)^{-\eta}$$ **Equation 2-20**

$$\frac{C_y}{C_{a*}} = e^{\eta\left(1-\frac{y}{a*}\right)}$$ **Equation 2-22**

Where: C_{a*} = concentration at reference level a* (mg/l)

C_y	=	concentration at level y (mg/l)
κ	=	von Karman's constant (a value of 0.4 was assumed)
y	=	level above pipe invert (mm)
a*	=	reference concentration level (mm)
η	=	sedimentation parameter

Equation 2-20 and Equation 2-22 describe the vertical distribution of sediment concentrations in a 2-dimensional steady flow for the inner (close to invert) and outer (close to free surface) regions respectively.

The procedure requires a known concentration at the known level, a*, with a reference level of one quarter of the total depth (from pipe invert) recommended to be used in conjunction with the method. Although tests using a variety of recorded data sets have yielded impressive profile predictions, the reliance of the approach on a known concentration at a known height is highly restrictive to the wide-scale application of the equations. This has been addressed through the provision of a suitable relationship to estimate the concentration at one-quarter depth, although it is clear that the accuracy of the approach then relies on the accuracy of the method used to determine reference concentration. Given the accepted level of performance of current sediment prediction equations it may be inappropriate to carry out detailed sediment profiling techniques using a reference figure which is inherently inaccurate.

At present, it is likely that profiling such as this is only suitable where detailed data are available in locally concentrated or laboratory studies.

2.7.3.4.2 Arthur (1996)

The approach developed by Arthur assumes that the material moving near the bed during periods of low turbulence is a mixture of the broad range of particle types present in domestic sewage. Consequently, rather than attempt to represent the broad range of particle characteristics, the potential for near bed solids transport is determined using a combination of empirical and statistical factors based around the flow history of a location in conjunction with some basic assumptions made regarding the material transported.

$$Cv = -105.73 + 2.55\times10^{-3}\left(\frac{I_r TSSS}{D_r}\right) + 0.2023\left(\frac{y_o}{y_{max}}\right) + 47.808\left(\frac{\tau_o}{\tau_b}\right) + 120.45\left(\frac{\rho_d}{\rho_w}\right)$$

Equation 2-24

Where: Cv = the volumetric sediment concentration
I_r = the peak intensity of the last storm event (mm/hr);
TSSS = the time since the start of the previous storm event (hrs);
D_r = total depth of previous storm event (mm);
y_o = ambient flow depth (mm);
y_{max} = daily peak dry weather flow depth (mm);
τ_o = average shear stress (N/m^2);
τ_b = bed shear stress (N/m^2);
ρ_d = dry density of bulk bed material (kg/m^3);
ρ_w = wet density of bulk bed material (kg/m^3).

Equation 2-24 (above) was developed using observations taken at three sites in Dundee. The sites were selected as they offered varying hydraulic and sediment transport characteristics, with two sites used for model development and the final site used for verification purposes. The performance of the final model was then compared with that of several other leading bed transport models.

The performance of the Arthur relationship was found to best match observed transport rates at all sites. However, as these locations were used in model development, this may not indicate overall best model performance. For a true assessment of model performance, the relationship should be compared against other models over a range of locations and hydraulic conditions. Although it can be argued that the empirical approach developed here can only produce site specific relationships, the consideration of bulk sediment properties is in fact no more likely to be erroneous than the assumption of many physical models of a single particle size to represent all sediment types.

2.7.3.5 Total-load Relationships

Total load equations determine the total quantity of material in motion under a given hydraulic condition. This includes all material moving as wash-load, suspended-load and bed-load. This has the advantage to practitioners of being easier to apply than a combination of equations for each transport mode. It does however present difficulties to researchers regarding the measurement of the total load as the complexities of sediment profiles and two phase motion must be addressed.

A further limitation of a total-load relationship is the selection of an appropriate representative particle. As both bed and suspended loads are represented by a total-load approach, a wide range of particle characteristics is implied. It is therefore unlikely that a single particle size and density can be used to accurately represent all particles transported under a range of conditions.

The relationships developed by Ackers and White (1991) are probably the most broadly tested set of total-load equations currently used. The various revisions and updates of the procedure have resulted in the testing of particle sizes from 0.04 mm to 132 mm. Within the Ackers and White procedure, particles are characterised using the dimensionless grain parameter D_{gr} (Equation 2-26). The particles used in the development of the Ackers and White relationship relate to a range of D_{gr} values from 1.1 to 132.

$$D_{gr} = \left[g\left(\frac{SG-1}{\nu^2} \right) \right]^{\frac{1}{3}}$$ **Equation 2-26**

Where: Dgr = dimensionless grain parameter
g = acceleration due to gravity (m/s^{-2})
SG = specific gravity
ν = kinematic viscosity (m^2/s)

2.7.4 Sediment Transport Model Testing

The most significant study to date which has tested the widest range of methods and their applicability to sewer sediments was carried out by Ackers, Butler and May. The objective of their research was to develop a standard method for the hydraulic design of sewers to control sediment problems and to produce guidance for practicing engineers. The guidance took the form of CIRIA Report 141 (Ackers et al., 1996) and utilised a range of sediment prediction methods to produce a new set of design tables for sewer design. The report includes details of the assessment of the various transport methods. Wherever possible, the recommendations of CIRIA Report 141 have been followed in this study.

2.8 Deposition Prediction

Methods for analytically predicting sedimentation in sewerage systems are still embryonic. It has been shown that deposition in sewers generally occurs during periods of dry weather and also during decelerating flows when storm runoff is receding. Locations of deposition have also been shown to correspond strongly with structural and hydraulic discontinuities such as joints, gradient changes and junctions. Due to the random nature of localised discontinuities such as poorly fitting pipe joints, it is believed that deposits associated with these discontinuities will be very difficult to predict.

It is also known that a pipe's propensity for sediment deposition will vary according to its relative location within the sewer network (i.e. the relative type of flow input), and the physical pipe characteristics such as size, shape, gradient and condition. Settlement and deposition will therefore occur at a rate depending upon the flow field, the nature of the particles and the concentration in suspension and/or near the bed. The importance of rapidly varied flow effects for transport and deposition in large sewers has been observed in many studies.

In most sewers, the DWF patterns provide 'fresh' supplies of solids from upstream 'stores', often laid down during low night-time flows. Storms provide a more random source of sediments and disturbances. The combination of the dry weather and storm conditions can cause deposits to become layered and mixed over time due to these interacting processes and on-going biochemical reactions. Hence deposited beds in sewers are heterogeneous and can exhibit thixotropic characteristics.

In recent years, varying approaches to the prediction of sedimentation in sewers and drainage systems have started to develop. Most recently attempts have been made to use the new generation of commercial sewer flow quality models to determine sediment deposition (e.g. MOUSETRAP: Mark et al, 1996). However, these models have been shown to be very sensitive to small changes in critical data, such as particle characteristics and bed strength (Gent et al, 1996) and therefore almost impossible to verify. The standard UK sewer flow quality model, InfoWorks now claims to be able to predict deposition and erosion via a two-fraction sediment model. Parallel studies in Perth have illustrated the futility of attempting to use the previous one-sediment fraction model for this (Ashley et al, 1997). The HydroWorks two-sediment fraction model was tested as part of this study by carrying out a continuous simulation of 28 days using the Murraygate sewer as a test location and later using InfoWorks and the Lugar area as a long-term test. The results from this long-term testing are discussed further in Appendix B.

Sewer hydraulic models are also of use in order to provide the hydraulic inputs which can be used to characterise sedimentation. Methods based on mapping boundary shear stresses have been used in the past (Pisano et al., 1979, Gent et al., 1996, Mark et al., 1998). The applications of these methods have varied slightly in technique but have all essentially involved the prediction of flow velocities and resulting boundary shear stresses. These boundary stresses are then compared to a predetermined critical value to classify the risk of sedimentation in each pipe

Studies carried out principally by Mark (Mark et al., 1996; Mark et al., 1998) have used this approach to determine locations and quantities of sedimentation over a number of catchments. However, although predicted sediment depths are claimed to be produced by the analysis, no published verification of these figures has been provided to date. Instead it is suggested that this procedure can only be used to compare the performance of two or more pipes within a system and hence highlight locations more likely to suffer from deposition problems. Insufficient verification data have been published from these studies to allow a thorough assessment to be made.

An assessment of each of the available deposition prediction techniques has been carried out in order to assess their scientific basis, ease of application and suitability of each approach to be used within the context of a sediment management strategy. The details of the leading techniques are given in the following sections.

2.8.1 Gent & Orman, 1991 - An Analytical Approach Using Sewer Hydraulic Models

The Gent and Orman method uses a hydraulic model in order to produce flow depths and velocities throughout the entirety of any catchment. The basic assumptions of the procedure are:

- Dry weather flow conditions are treated separately from storm conditions.
- Peak dry weather flow conditions occur daily so any sediment eroded during dry weather flow is assumed to be non-cohesive as it is unlikely to have started to agglutinate. Sediment eroded in storm conditions acts as cohesive sediment because it has had time to agglutinate. Once eroded it becomes non-cohesive when transported and deposited.
- Surface sediment wash-off will only occur in storm conditions. Therefore for sediment deposition to occur in dry weather, erosion of sediment must have taken place in a pipe upstream.
- If erosion of sediment occurs under dry weather flow then any sediment deposited during storm conditions will be quickly re-eroded.

It is proposed that the analysis should be carried out in a predetermined order of calculation as shown in Figure 2.43 and described further in the following sections.

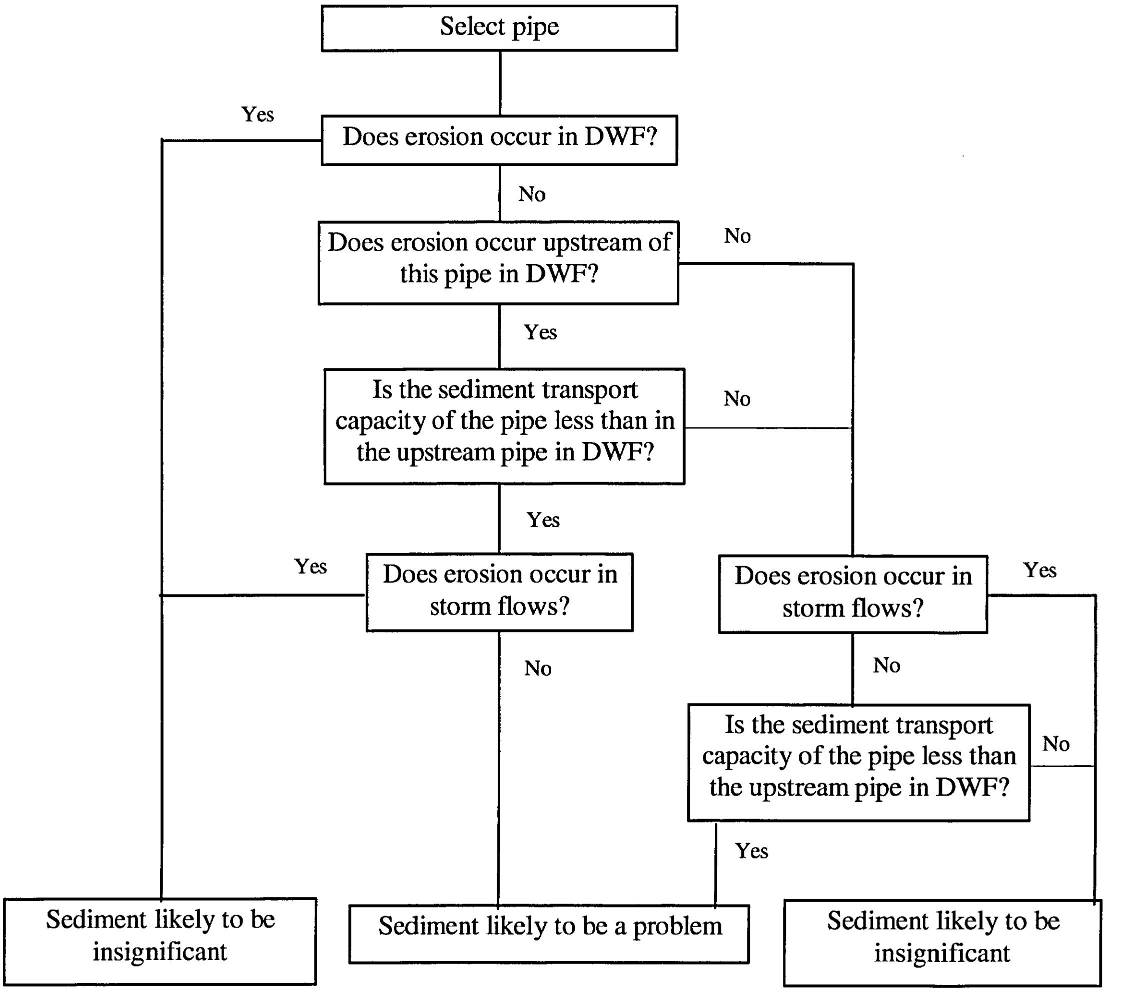


Figure 2.43 -Flow diagram of Gent & Orman Procedure to Locate Sediments

Dry Weather Flow Criteria

Does erosion occur in dry weather flow?

This criterion is examined by considering which pipe lengths will achieve velocities in excess of the critical erosion velocity during peak dry weather flows. The proposed procedure involves the 'looking-up' of design charts in order to determine if erosion will take place. The charts are based on the assumption of normal depth of flow, and use a combination of the Colebrook-White flow equation and Ackers-White sediment transport theory. This is carried out for all pipes in a system, and the results plotted onto a plan of the system. It is also suggested that the charts can be used for the design of sewer pipes to be 'self-cleansing'.

Does erosion occur upstream in dry weather flow?

If the critical erosion velocity is not achieved in any particular pipe under dry weather flow, then sedimentation is only possible if erosion has taken place in a location upstream. This stage in the procedure results from the assumption that sediment sources under dry weather flows are purely in-pipe. The locations of any upstream areas of erosion are identified by the use of the plan produced in the previous stage.

Is the sediment transport capacity of the pipe less than in the upstream pipe in dry weather flow?

If the first two conditions have not been satisfied, deposition is still only possible if there has been a reduction in the sediment transport capacity from upstream conditions. For each of the pipes around the pipes highlighted as potentially problematic in stages 1 and 2, the sediment transport capacity under peak dry weather flows should be evaluated. The transport capacity of the upstream pipes should be taken as the minimum value up to the point of erosion (working in an upstream direction from the pipe considered). The suggested method of determining the transport capacity of the dry weather flows is by the use of a simplified version of May's theory (May et al., 1989). Those pipes where dry weather flow deposition is predicted should be marked onto a plan of the system.

Storm Flow Criteria

Does erosion occur in storm flows?

It is assumed that deposits which are not readily erodible by dry weather flows will consolidate and develop an increased shear strength with time. A design shear stress of 9 N/m^2 has been assumed for this procedure, as this is believed to offer a factor of safety of around 1.3 over the accepted critical value for lightly consolidated sediment (Nalluri and Alvarez, 1992). Similarly a design shear strength of 2.5 N/m^2 has been assumed as a suitable design value for recently deposited sediment.

In order to determine the frequency of occurrence of the flows required to produce erosion of this material, it is recommended by Gent & Orman that two events from a time series rainfall set should be selected. These are the 15th event in an annual series, to be used with the design shear stress of 9 N/m^2, and the 90th event for use with the 2.5 N/m^2 strength value.

Where the design values of shear stress are exceeded, it is believed that sedimentation is unlikely, even though deposition may take place during dry weather. In locations where the design shear stresses are not exceeded and dry weather deposition has been shown to be likely, it is then necessary to consider the transport capacity of the pipes under storm flows.

Sediment Transport Capacity Under Storm Flows

If the erosion velocity is not achieved in either dry weather or storm flows, then deposition may occur. As with dry weather flows, this will only happen if there is a decrease in the sediment transport capacity downstream. However, as sediment is continually entering the system during storm flows (model assumption), it is not necessary to consider the erosional characteristics of the upstream pipes.

The procedure proposes that the sediment transport capacity of the entire system should be evaluated, with the following exceptions;

- where erosion occurs under storm conditions (as identified in the previous stage;
- where erosion occurs under dry weather flow;
- where deposition occurs in dry weather flow.

The proposed method requires design tables, giving Cv values for half full and full depths of circular pipes of varying diameters under different flow velocities. Locations where the sediment transport capacity of the flow is reduced under either storm considered should be highlighted as areas of likely sedimentation.

Verification of Procedure

The procedure was applied by Gent and Orman to a small, self-contained test catchment in order to ascertain the accuracy of the methods used. Flow velocities were calculated from the peak discharges and the corresponding depths on the hydrographs as at this time errors were known to exist in the velocity calculations of the WALLRUS hydraulic model.

The procedures were followed using the design charts produced. It was claimed that all areas of 'major' deposition were correctly predicted, and that in general the method slightly under-predicted the extents of this deposition. It is impossible to comment objectively on the accuracy of the verification, as details of the measured data were not included within the results. However, inconsistencies were observed in the graphical plot of results, with the final plot of predicted sediment deposition not agreeing completely with the plots produced at the previous stages.

2.8.2 Pisano et al., 1979

This method was proposed as part of a pollution control initiative carried out by the U.S. Environmental Protection Agency. The proposed model was developed in order to address the following criteria.

a) identify areas of extensive collection systems subject to high degrees of sediment deposition;

b) indicate the relative degrees of deposition among different parts of the system;

c) provide an indication of the order of magnitude of daily deposition throughout the system.

The method is mathematically very simple and does not require any hydraulic driver to provide the flow data. Although this makes the procedure simple to use it is limiting in that only 'average' dry weather flows can be applied.

The susceptibility of any particular section of pipe to sedimentation is estimated by a comparison of the bed shear experienced under average dry weather flow, with a critical bed shear erosion criterion. The critical bed shear was established by considering Shields' erosion criterion (Shields, 1936) and Hughmark's minimum shear stress for maintaining suspension (Hughmark, 1961). This resulted in the following relationship:

$\tau_c = 0.02\ d^{2/3}$ **Equation 2-27**

Where: τ_c = critical boundary shear stress (psf)
d = particle diameter (ft)
N.B. This is an empirical relationship giving units of psf.

This resulted in the following relationships being developed for the sedimentation efficiency of any particular section of pipe:

$$Z = 40\left(\frac{\tau}{0.004}\right)^{-1.2} \qquad \text{for } \tau > 0.004 \text{ psf}$$ **Equation 2-29**

$$Z = 40 \qquad \text{for } \tau \leq 0.004 \text{ psf}$$ **Equation 2-31**

Where: Z = the percentage of suspended solids in dry weather flow deposited if wall shear stress is less than τ_c.

The general procedure as provided by the original USEPA report is as follows:

1. Compute cumulative upstream population for the end of each link.
2. Compute average and maximum daily dry weather flows from consideration of population and contribution per capita.
3. Compute shear stress for each link associated with the maximum daily flow.
4. Calculate the dry weather deposition rates for each link - Z_i.
5. Determine the suspended solids loads developed in each link from consideration of population and contribution per capita - ZL_i.
6. Starting at the uppermost link, compute the amount of input material that will deposit - A_i x ZL_i.
7. Search the list of downstream links for the deposition rate greater than the rate at the link where the load is initially generated, and compute the amount deposited as the jth link from the ith component input load using $(Z_j - Z_i)$ x ZL_i.
8. Continue searching the list of downstream links for a deposition rate Z_k greater than Z_j and compute the deposition ant the kth link from the ith component using $(Z_k - Z_j)$ x ZL_i.
9. Set $Z_k = Z_j$ and repeat steps 7 & 8 until the complete list of downstream links is completed.

10. Start with the next uppermost link and repeat steps 6 through 9 while maintaining a running sum of all the deposited loads in each link from previous iterations.
11. Sequentially proceed downstream until all components are completed.

Method Verification

The model was verified by applying the procedure to four test segments from within a single catchment. The test segments used were chosen as they had been utilised in a previous study concerning the flushing of sediment deposits, and therefore the results from these tests were used as a measure of the mass of deposited sediment. Alternative methods of measurement were also taken, primarily involving the 'scraping' of a one foot length of pipe, and applying the mass of sediment removed over all lengths affected by sedimentation.

The tests were carried out for various assumptions of per capita waste flowrates. Table 2.10 shows that a limited agreement of data (with set B) was achieved when considering a per capita waste flowrate of 60 gpcd, with the exception of Port Norfolk. The inconsistency in the measured data at this site (and to a lesser extent, the others), brings into question the accuracy of the flushing method of measurement. Although the flushing efficiency was calculated at 76% during flush tests, a very different set of efficiencies is achieved when considering the scraping data. For the three streets sampled, flushing efficiencies of 84%, 48% and 34% can be calculated, thus giving an 'average' flushing efficiency of 55%. This large variation suggests that at least one (and perhaps both) of the methods used to 'measure' the quantities of deposits may be inaccurate.

Street	Deposition Model Predictions (kg/d)				Measured Data (kg/d)		
	60 gpcd	100 gpcd	150 gpcd	200 gpcd	A	B	C
Templeton	3.49	2.60	2.09	1.79	2.54	3.34	3.01
Shepton	1.69	1.28	1.02	0.87	1.31	1.73	2.71
Port Norfolk	3.33	2.54	2.01	1.71	1.30	1.71	3.80
Walnut	2.88	2.99	2.41	2.02	1.72	2.26	-

Measured Data Method:

A- Flush removal rates for one day of build-up

B- Flush removal rates subjected to a flushing removal efficiency of 76%.

C- Application of scraping tests for each street.

Note: gpcd = gallons per capita per day

Table 2.10- Verification of US EPA Method

The lack of reliable measured data makes it difficult to draw conclusions on the accuracy of the proposed method, although all figures are of the same order of magnitude. The method may be of best use as a comparative tool depending on the consistency of the results achieved on a larger scale. Inspection of the limited data-sets presented, show the order of depositional significance to vary with the per capita wastage flow rate, but again a variation is shown in the measured values collected by the two methods B and C.

Although the verification work carried out is inconclusive, it does demonstrate that using sensible, estimated parameters, results given by the method are at least comparable to field measurements both qualitatively, and to a lesser extent quantitatively. The percentage removal rate is a simple calculation procedure which could be refined with greater data collection. As all of the relationships were derived empirically, the site-specific nature of the method will require to be tested by further application.

The approach was subsequently refined in German studies for modelling purposes (a model called THALIA), and applied by WRc to some English catchments with mixed success (Crabtree et al, 1991). The USEPA method was applied to two test catchments in Dundee (the main Dundee central area catchment and the Upper Perth Road) (Goodison & Ashley, 1992). The results from the overall model indicated that some 138.5 kg/day should deposit within the Interceptor sewer catchment. This figure comprises some 14% of the total daily input to the system (1013.9 kg), based on the measured per capita daily production of suspended solids of 60g. Interestingly, the daily SS load of 60 g/h day measured in Dundee (summer days) corresponded precisely to the US data. The figures for Dundee were for end-of-pipe and thus ignored:

(i) gross solids - not sampled by small bore samplers;

(ii) bed-load - up to 20-30% of total solids in transport;

(iii) the *'a priori'* DWF solids deposition within the system (as estimated by the regression equation)

The gross solids load inputs cannot at present be quantified, but the effects of ignoring (ii) and (iii) above suggest that some 40-50% more solids are being generated at the inputs to the system, than the measured 60g/h day.

Application of the method to the Perth road sewer network in Dundee was made using the following data (Goodison & Ashley, 1992):

- population: 0.44 persons per metre length of sewer (total 2500)
- peak (winter) DWF: 18l/s
- rate of solids generated per head of population: 70g/hday (suspended solids measured at outfall)

The full EPA methodology was applied - which entails working successively down the sewer network - as upstream solids loads transported into successive downstream sewers would obviously influence the transport and deposition therein. Some correspondence was found between predicted and observed deposits. Work using this method was subsequently abandoned as the theoretical base of considering sanitary inputs only in dry weather was believed to be flawed.

2.8.3 Lin & Guennec (1996)

Lin and Guennec utilised classic bed load theory in order to predict deposition levels in the number thirteen trunk sewer in Marseille. The proposed model is for dry weather only, in steady, uniform flows. The calculations of hydraulic and sediment behaviour interact with updated inputs being used at each calculation timestep. Updates of bed slope, energy gradient, Froude number and cross section properties are used for the hydraulic calculations, and updates of sediment transport capacity, bed level and sediment particle diameter are used for the calculation of deposition.

The determination of deposition is calculated as follows;

1. The volumetric bed load transport rate under equilibrium conditions ($q^*_{s,k}$) is calculated using Meyer-Peter equation (Meyer-Peter & Muller, 1948).
2. The local solid discharge rate from the bed ($q_{s,k}$) is determined using the Daubert-Lebreton law (1967).
3. If $q^*_{s,k} > q_{s,k}$ erosion will exist, if $q^*_{s,k} < q_{s,k}$ deposition will exist.
4. Determine the ability of the bed to provide or accept solid particles for either erosion or deposition. This is done using Gessler's theory to define a 'mixing layer' (Little and Mayer, 1972).
5. The variations in sediment bed are calculated using the following equation:

$$\frac{\partial Q_s}{\partial x} + \frac{\partial (Cv_b, A_b)}{\partial t} = 0$$ **Equation 2-33**

where: Q_s= total bed load discharge;
Cv_b= the volume fraction of solids in the active mixing layer;
A_b= cross-section of the deposit

The procedure was calibrated using the upstream reaches of the Marseille trunk sewer, and verified using a section of sewer lower down the same sewer. Verification results show a high degree of correlation between observed and predicted values over a period of 1000 days. It was also noted that the distribution of the grain size of the bed was also predicted accurately.

The accuracy of results may have been influenced by the choice of verification site and its proximity to the calibration site. The approach is also very limited, as a steady flow rate is assumed, therefore the important influence of time varying flows and storms cannot be included in the analysis.

2.8.4 Laplace et al., (1995)

The work of Laplace centred on the development of a model relating rainfall characteristics with the quantities of sediment deposited in the same trunk sewer used in the Lin and Guennec study. Within this regression analysis, rainfall characteristics are described in three main ways:

- depth of rainfall in the previous 24 hours;
- antecedent dry weather period;
- maximum rainfall intensity.

A series of coefficients of correlation were determined for each of the above parameters. Using the resulting regression equation, a reasonable comparison of calculated and deposition volumes was made. However, as the coefficients relate to rainfall and catchment characteristics (via antecedent dry weather build up) the application of this method to other sites would be meaningless without the recollection of huge quantities of data. However, the method has proved to be a useful planning tool within the catchment used.

2.8.5 Laplace (2001)

In an attempt to remove some of the site specific elements of previous studies, Laplace characterised historic data according to key variables to be used in the development of a globally applicable conceptual model for predicting pipe deposit and sediment trap fill rates

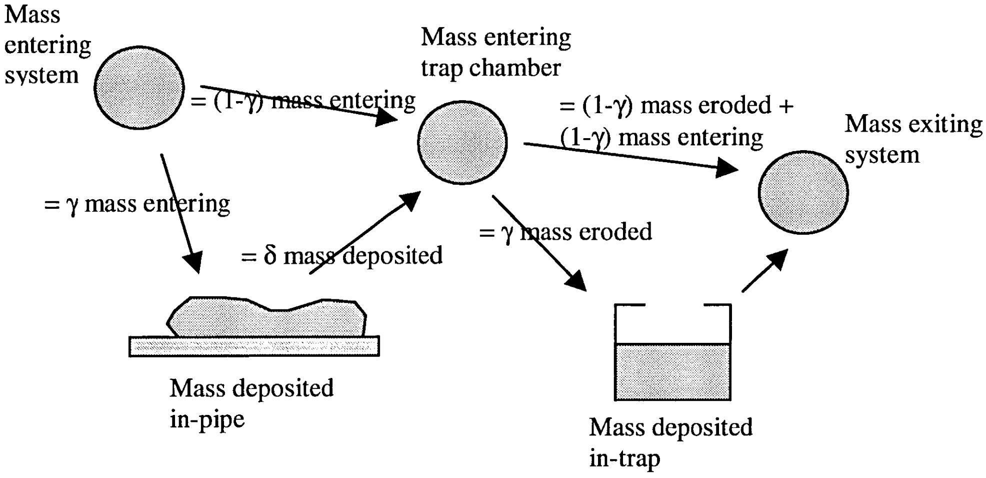


Figure 2.45 - Schematic of the Laplace conceptual deposition model

Figure 2.45 (above) shows a schematic representation of the model. A simple multiplier is used to determine the masses of sediment depositing and eroding. These factors are derived from the Marseille study and are:

γ (pipe deposits) = 0.09

γ (trap deposits = 0.18

The ultimate mass of pipe deposits is limited in line with previous study findings at 300 kg/ha of contributing area.

The resulting model uses the data of previous studies (for rate of solids production, average dry weather deposition rates and erosion rates) and weights the predictions of sediment deposition according to:

1. Catchment area;
2. Percentage of impermeable area;
3. Rainfall depth.

The deposition functions are assumed to result in an asymptotic decay of deposition rate for dry weather flows and assumed erosion for the rainfall events. These assumptions are taken from the studies of the number 13-trunk sewer in Marseille (Laplace et al., 1995). The model operates using a daily timestep and once successfully calibrated, has been shown to provide a reasonable representation of sediment trap filling over a prolonged duration (Figure 2.46).

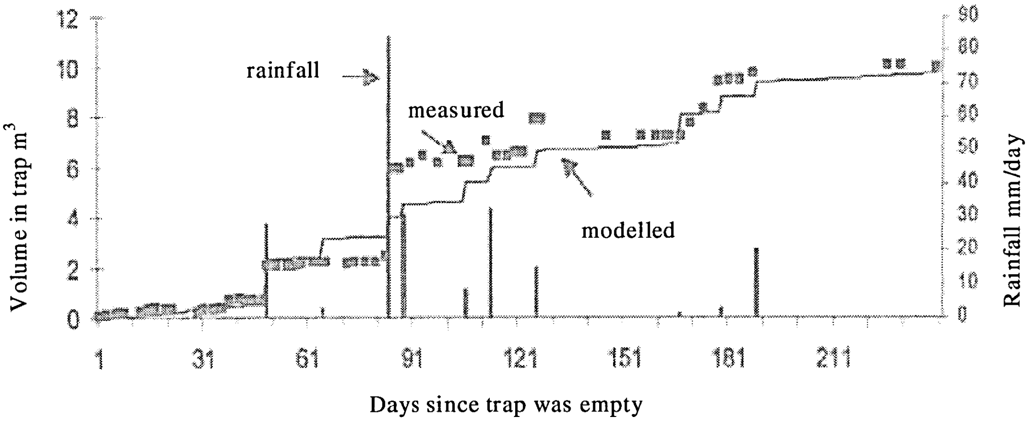


Figure 2.46 - Comparison of measured and modelled sediment trap volumes (after Laplace, 2001)

The model is simple in concept and is easy to apply. However, the majority of the simplifications and assumptions are based solely on the findings of a single previous study. Although calibrations have proved successful in applying the method to a further location, it is likely that the approach will not prove as successful if catchments of different characteristics are used (e.g. gradient and solids loadings). This belief results from the comparison of the assumptions used, with alternative data. For example, erosion may not take place during rainfall, with severe deposition noted at some sites during rainfall events. Additionally, under the conceptual model, only the eroded sediments are believed to contribute to trapped volumes. In reality, a number of further sources including suspended solids may contribute depending upon ambient hydraulics and catchment characteristics.

2.8.6 Sewer Flow Quality Modelling (InfoWorks v 5.01)

The use of sewer flow quality models to determine sediment transport, deposition and quality impacts is becoming increasingly common. However, a fundamental analysis of the modelling techniques is required despite the apparent widespread acceptance of the models and the impression of accuracy that they give. A summary of the techniques is provided in the following sections. An assessment of the two-phase sediment model can be found in Appendix B.

2.8.6.1 Wash-off Modelling

The wash-off model represents the build-up of surface sediments during dry weather, followed by the mobilisation of these particles during storm events. During these events the particles are transferred into the wastewater system.

The key parameters that must be defined by the modeller to allow the modelling of this process are contained within the "Surface Pollutant Editor". In this way, the mass of initial surface sediment can be defined along with the rate of sediment build-up during dry weather. This build-up rate uses an assumed exponential decay function used in many models (Bertrand-Krajewski et al., 1993). The model assumes that over time, an equilibrium is reached between the supply of solids and their removal. Coarse testing of the build-up parameters has shown the model to be significantly more sensitive to the assumption of the initial mass of solids on the catchment than to the alteration of the sediment build-up factors. It is therefore recommended that great care should be taken when setting the initial mass of sediment present on the catchment surface.

The testing carried out within this study assumed no initial catchment sediments. This is principally because only long-term simulations were used. For event by event analysis, the initial surface condition is of far greater significance. In this case it is recommended that a low initial level of surface sediments is assumed, with a prolonged dry weather flow leading in to the event. The storm event prior to the event of interest should then be used along with the appropriate length of antecedent conditions to define the initial conditions for the storm of interest. This approach allows a measure of the time history on the catchment surface to be taken into account.

The actual mobilisation of the surface sediments is calculated using the chosen hydraulic run-off routing model, the rainfall intensity and the mass of sediment on the catchment surface. The initial erosion of the sediment particles is calculated using a simple function of rainfall intensity and the mass of sediment available. The latest release of InfoWorks allows modellers to define the maximum allowable erosion rate. This option should be used with caution and only by experienced sediment transport modellers.

$$\frac{dMe}{dt} = Ka.M(t) - f(t)$$ **Equation 2-35**

Where: Me = mass of sediment eroded
M(t) = the mass of surface deposits
Ka = the erosion / dissolution factor related to rainfall intensity

Following this calculation, the quantity of sediment washed-off is determined on the assumption that the pollutant flow at any subcatchment outlet is proportional to the quantity of pollutant in suspension in the storm water present on the catchment. Consequently the output from the runoff routing model is used to determine the masses of sediment washed off according to Equation 2-37.

$$Me(t) = K.f(t)$$ **Equation 2-37**

Where: Me(t) = mass of sediment in suspension
K = the linear reservoir coefficient (from Desbordes)
f(t) = the pollutant flow

It is in this area of the wash-off model that one of the recent changes becomes significant. Previous versions of the software did not match the routing models used in the hydraulic and quality calculations. The effects of this were investigated by Bouteligier et al. (2002). This work found that the previous assumption resulted in sediment wash-off occurring up to 4 hours after run-off had stopped. The implications of this for a single event are clearly significant. The current software version allows the user the option of matching the quality and hydraulic runoff models.

It is also significant for the modelling of two phases of sediments, that only sediment fraction 1 (SF1) can be washed off from the catchment surface. This is significant as it means that SF1 must be used to represent coarse solids. This has wide-reaching implications regarding the modelling of sediment deposition as it is the coarse particles that play the dominant role. These implications are not stated with sufficient clarity in InfoWorks guidance notes.

2.8.6.2 Gully Pot Modelling

Within the InfoWorks models, gully pot modelling has no impact on the transport of sediment from the catchment surface, with only dissolved pollutants introduced at the gully pots. Early model testing has indicated that the overall sensitivity of the model to the effects of modelling the gully pots is low (Wallingford, 2003). As the primary focus of this study is the modelling of sediments, further investigations were not undertaken.

The impact of ignoring gully pot deposition and erosion processes has not been ascertained. However work carried out at the University of Bradford has indicated a low level of sensitivity of long term depositional patterns to gully cleaning practices (Gouda et al, 2003). Single event sensitivity has however been shown to be much greater, with Butler and Karunaratne (1995) testing the trapping efficiency of gully pots over a range of experimental conditions. This work showed a wide variability in gully retention according to the flow rate through the gully pot and the mean sediment particle size.

2.8.6.3 In-pipe Sediment Transport

At each modelling timestep, InfoWorks determines hydraulic and quality parameters at each calculation point within each node and conduit of a system. Until recently (Version 5.0), InfoWorks has used solely the theory of Ackers and White to determine sediment concentrations and levels of in pipe deposition. As the validity of transport modelling assumptions have been questioned over time, a range of additional features and modifications have been made to the InfoWorks software which can lead to some confusion over the actual mode of operation of the transport model.

Ackers-white transport relationships were initially developed for granular material tested in a rectangular, open channel flume. Since this time however, the original relationships have been modified to increase their applicability to drainage sediments (May, 1993; Ackers, 1991). However, a number of concerns of the use of Ackers-White persist.

- Sewer sediments can exhibit significant cohesive properties, rendering the use of Ackers-White to determine cohesive sediment erosion inappropriate;
- The range of particle sizes over which Ackers-White relationships were developed are generally more coarse than those found in dry weather solids;
- Initial use of the Ackers-White relationships was found to over-predict sediment concentrations.

These limitations result in a number of consequences for sediment modelling. The application of Ackers-White to determine sediment erosion, results in the use of inappropriate sediment characteristics in an attempt to represent the higher shear stresses required to erode cohesive material. The particle sizes used to achieve accurate sediment transport concentrations may therefore differ markedly from those used to achieve an accurate pattern of deposition, with the latter forced to an unrealistically high particle size and density.

The question over the applicability of the Ackers-White relationships to particles of the characteristics of dry weather solids is highlighted further by the fact that the default sediment characteristics provided within InfoWorks are in fact outside the range of applicability with a Dgr of 0.692. The impact of the use of the relationships outside of the range of validity was investigated by Bouteligier et al (2002).

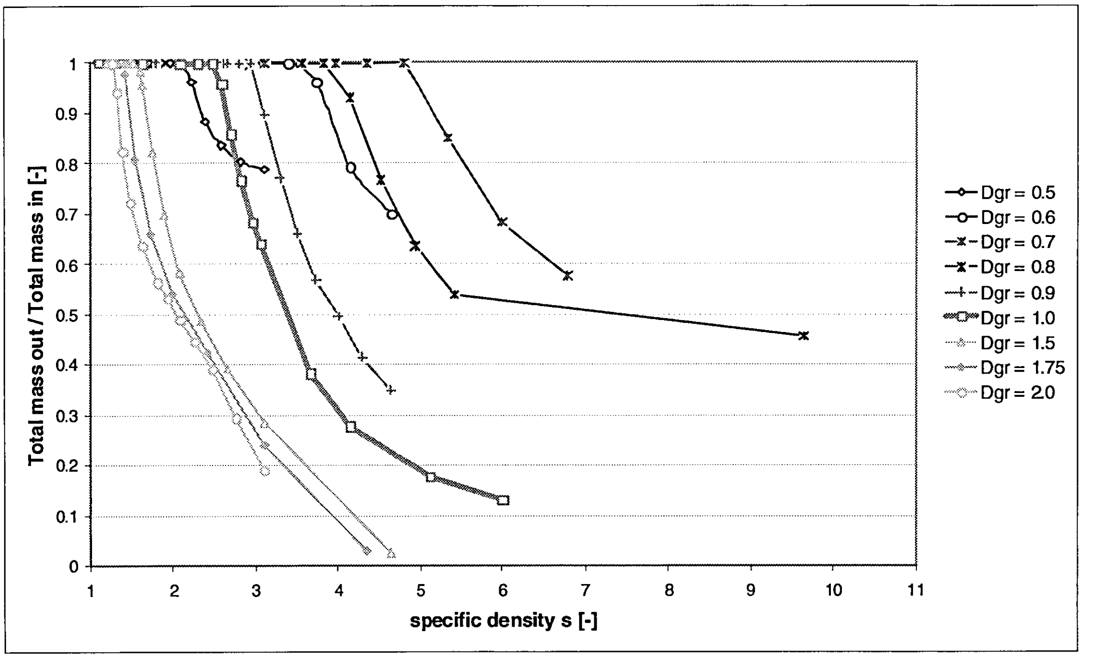


Figure 2.47 - Illustration of the effects of the use of Ackers-White outside valid range (from Bouteligier, personal communication)

Figure 2.47 (above) shows the implications of using the default values of InfoWorks along with Ackers-White transport relationships. The figure shows the total sediment mass exiting a sewer pipe relative to that entering, plotted against the specific density of the sediment fraction that is modelled. Each curve represents a particular dimensionless grain number (Dgr). It can clearly be seen that for a Dgr greater than 1, a steady pattern of reducing deposition with reducing Dgr results. However, as Dgr reduces to below approximately 0.7, the behaviour of the curves changes dramatically. The change of behaviour of the equations at this point results in an inconsistent behaviour with a reduced sediment transport rate shown for smaller particles of the same density as larger particles.

This paradoxical behaviour has been addressed by the developers of InfoWorks through the limiting of Dgr to 1.0 at its lowest end. This results in an inconsistent hybrid equation being used whereby the Dgr value bears no relevance to the particle characteristics used in the model.

The issue of the over-prediction of sediment concentrations using Ackers-White has been addressed simply via limiting the calculated values. This coarse adjustment has been carried out at three separate times during the development of the sediment transport module within InfoWorks. Each time, the adjustment has been made to a different limiting level. In addition to this, the total quantity that can be modelled is dependent on the number of sediment fractions represented in the model (Version 5.0), as the limit is set separately for each fraction. Consequently, when using two sediment fractions, twice the total mass of sediment can be modelled than when using only one sediment fraction. This array of complications results in a series of inconsistencies which depend upon the version of the model used and the settings within that model. The possibility therefore arises that previously calibrated models are no longer valid when used on an updated platform.

A combination of all of the difficulties associated with the Ackers-White relationships has resulted in the inclusion of two alternative transport models in InfoWorks 5.0. These are:

- The KUL model – an excess shear relationship which uses both a critical erosion and deposition shear value;
- The Velikanov model – A turbulent energy based model which uses a minimum and maximum sediment concentration for a level of turbulent energy to determine whether erosion, deposition or neither process occurs.

Both of the above models are simple in concept, with the KUL model very similar in structure to the modified USEPA method developed in Chapter 4 of this thesis. Detailed analysis of these models has not been carried out to date as a result of the recency of their inclusion. However, the similarities of the KUL model to the methods developed within this study suggest its increased applicability for determining sewer deposition and erosion.

Chapter 3 Field Investigations

3.1 Introduction

In order to devise improves sediment prediction models, it is necessary to understand and measure the behaviour of sediments in real drainage systems. The field investigations carried out in this study required the observation of a variety of sediment behaviours. As the models developed in this study focus on sediment erosion, deposition, transport and trapping mechanisms, pertinent field data were required for each of these behaviours. In addition to this, a general sampling programme was undertaken to allow site and sediment characteristics to be assessed.

Three locations were studied in detail:

- Dundee City Centre & Constable Street invert trap;
- Upper Dighty catchment and Baldovan Rd - Claverhouse invert trap;
- Lower Forfar catchment and Forfar 900mm trunk sewer invert trap.

Table 3.1 (below) shows a summary of the variety of data collected at each site and the principal purposes of each data collection exercise.

Characteristic	Sites	Purpose of Measurement	Period of Measurement
Rainfall	Dundee City Centre Forfar	Verification & driver for rapid hydraulic simulator	Sept 1998 – Sept 2001
Sewer flows	Constable St invert trap Baldovan Rd invert trap Forfar invert trap Murraygate interceptor sewer	Hydraulic characterisation & model verification	Sept 1998 – Sept 2001
Sediment transport rates (suspended & bedload)	Constable St invert trap Baldovan Rd invert trap Forfar invert trap Murraygate interceptor sewer	Material characterisation, trap model inputs, sediment model verification.	Intermittent measurement

Sediment trap fill volumes	Constable St invert trap Baldovan Rd invert trap Forfar invert trap	Trap performance assessment, trap model verification	Intermittent measurement
Sediment bed level surveys	Murraygate interceptor sewer Historic data sets Forfar 900 mm trunk sewer	Bed gradient assessment, erosion model verification, pipe performance assessment, deposition prediction verification	Apr 2000 – Mar 2001 continuous measurement & intermittent measurement
Sediment bed samples	Murraygate interceptor sewer Dundee system Forfar 900 mm trunk sewer Forfar system	Bed material characterisation	Apr 2000 – Mar 2001 continuous measurement & intermittent measurement

Table 3.1 - Summary of field investigation framework

The following sections provide details of the locations investigated, the methods used and the data collected during this programme of research.

3.2 Catchment Details

In order to reduce any site specific bias to the models developed within this study, the field sites were selected to provide a range of conditions to enable the development of general rules for sediment modelling.

Initially field sites were selected in the Dundee area in order to build on the experience of previous Dundee investigations. However, the development of sediment related operational problems in the nearby town of Forfar resulted in the extension of the study to two further sites.

3.2.1 Dundee Drainage Network

The drainage system serving the City of Dundee and its surrounding areas can be considered to be divided into four main catchment areas (see Table 3.2). The majority of fieldwork carried out for this investigation was located in the City Centre and Dighty catchments. The four catchment areas are also shown in Figure 3.1.

Catchment	Area	Population
Invergowrie/Riverside	1396 ha	28,760
Dundee City Centre	961 ha	44,900
Stannergate	1135 ha	29,950
Dighty	1648 ha	42,600

Table 3.2-City of Dundee Catchments

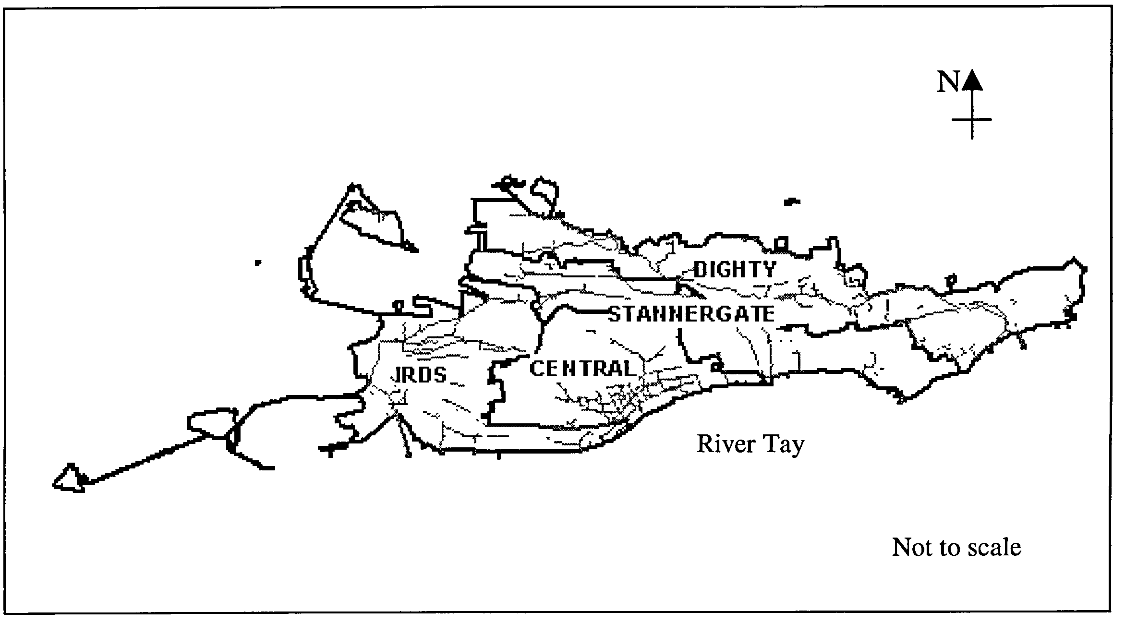


Figure 3.1 – Dundee catchment areas

Historically (and during these investigations), each of these drainage catchments discharged independently to the estuary of the River Tay. However, the implementation of the Wastewater Treatment Directive, 1991 (CEC, 1991) required that all sewage flows must be treated to a given standard prior to discharge. As a result of a shift of funding policy within the UK water industry, the scheme selected as a solution to the treatment requirement involved a Public Private Partnership (PPP). Within this scheme, a private consortium is responsible for the treatment and conveyance of flows derived from the wastewater network. The PPP consortium is also responsible for the construction and capital costs of additional infrastructure.

The operation and responsibility of the actual drainage network in this scheme is still retained by the water authority.

Flows from the various catchments in the Tay area (Invergowrie-Riverside, Central, Stannergate, Dighty, Carnoustie and Arbroath) are transferred via a large diameter pumped main to large-scale treatment facilities located at Hatton (west of Arbroath). Prior to the operation of the new regime, agreements were reached on acceptable ranges of flows and concentration levels to be received by the privately operated system. These agreed values are then factored into a complex tariff system to be charged to the water authority. Consequently, greater control of sediment movement may prevent high future costs incurred to the water authority as a result of breaches of these agreements.

3.2.2 City Centre Catchment

The City Centre catchment is predominantly drained by gravity via a combined sewerage system to a large egg-shaped (up to 1.8m in height) interceptor sewer, and the Dock Street sewer (up to 2.1m high). Figure 3.3 shows a schematic of the City Centre's principal sewers.

The City Centre system has steep collector catchments draining to a relatively flat upper river terrace level. This is where the large diameter Interceptor Sewer is located. On the lowest (river) level, a second large interceptor sewer runs along Dock Street and drains a portion of the lower part of the core business area of the city. Both sewers historically (and during this study) have discharged directly into the River Tay without treatment or tidal control. Currently however, both interceptor sewers drain to a pumping station, located at King George V wharf in the harbour area of Dundee.

As a result of the steep drop of the main interceptor from the upper river terrace, the alteration to the outlet condition has had few consequences for its performance. However, the Dock Street sewer has been previously heavily influenced by the tide, resulting in historical flooding problems in the town centre. It is anticipated that the recent alterations to its operation (including the construction of a new tunnel sewer) will improve these operational deficiencies.

The system pattern is mainly dendritic; however, flow patterns are complicated by the presence of more than 250 control gates used to divert flows around the system. These gates can cause loops to form within the system and, whilst offering flexibility of operation, can cause further complications unless closely monitored. Gate locations (settings) have tended to be considered by Dundee Divisional operating staff as inviolate and in practice the pattern is not altered unless for maintenance or construction works. The complexity of the gate system required the network to be simplified by further subdivision into 12 areas (Table 3.3) for the modelling of the central catchment.

Sub Catchment	**Pop. (x1000)**	**Area (ha)**	**Land Use**
Perth Rd.	4.80	164.9	T,I,P
Constitution Rd.	2.90	64.6	T,H,I,S,P
Hilltown	1.90	30.3	T,H,I,P
Dens Rd.	11.5	236.4	T,H,I,S,P
Dura St.	1.60	37.5	T,H,I,S,P
Albert St.	2.20	11.2	T,H,I,S,P
Hawkhill	0.28	6.8	T,I,P
Blackness	1.30	20.2	T,H,S,P
Polepark	5.80	159.9	T,H,I,S,P
Guthrie St.	0.42	19.3	T & I
Lochee	0.20	14.2	T,I,S,P
City Centre	~5.30	~25	T,S,P
TOTALS	37.24	909.6	

Land Use: T - Tenements/High Rise; H - Housing; S - Retail; I - Light Industry/Commercial; P - Park & Permeable Areas; H - Hospital

Table 3.3 - City Centre Sub-Catchments

Table 3.3 shows the subdivision of the city centre catchment into 12 areas chosen geographically and accounting for gate connections between these areas. Approximations were required for the City Centre population figures as a result of the wide variations throughout the day caused by commuting workers and visiting retail customers. All areas but the Perth Road and Lochee subcatchments drain to the main Dundee Interceptor Sewer.

The interceptor sewer itself starts as a flow diversion chamber in the City Centre. From this chamber, a gate can be positioned to either allow flows to continue along the interceptor sewer, or alternatively be diverted to run parallel to the interceptor sewer via the Dock Street Sewer. From the flow diversion chamber, the interceptor sewer runs in an easterly direction at an approximate gradient of 0.7% (Ashley et al. 1989). The interceptor sewer has been shown to suffer from problems of sediment deposition along a large proportion of its length, with deposits of 300 mm (~16% of the pipe height) not uncommon (Arthur, 1996). As part of a sediment control strategy, the interceptor sewer has a large on-line sediment trap installed at its head. However, the fill rate and therefore emptying of this trap has been seen to vary greatly with weather conditions, thus making planned, effective use of the trap difficult. Further details of the main interceptor sewer are given in Section 3.2.2.2.

The other main sewer, the Dock Street Interceptor, has an average gradient of approximately 0.0037 (1 in 270). However, the relative level of this sewer is such that the tide historically has filled it completely when high. This condition has been recorded within the system as far inland as Exchange Street in the City Centre (approximately 450m inland). The attendant low velocities are responsible for both sedimentation along the length of the sewer and for a build-up of fats and greases on the sewer walls.

A layout of the trunk sewers in the Dundee Central catchment is shown in Figure 3.3.

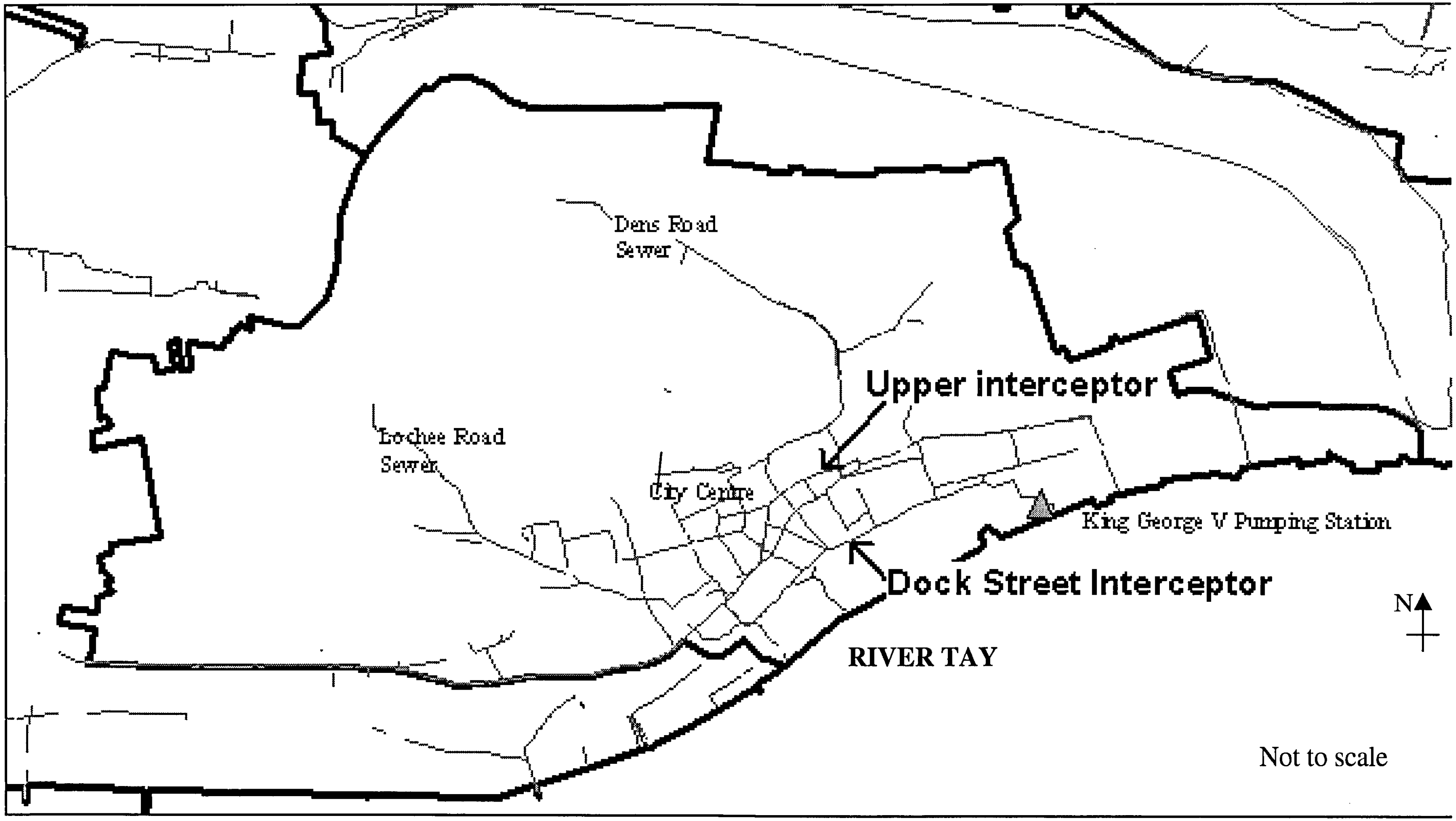


Figure 3.3 - Central catchment trunk sewers

3.2.2.1 Summary of Previous City Centre Studies

The drainage systems of Dundee have been subject to a wide range of studies. The most significant of these have been predominantly concerned with the movement and analysis of sediments in and around the Dundee City Centre sub-catchment.

Sediment Behaviour in Combined Sewers - A Programme of Research for WRc - Richard M. Ashley (1993)

The most extensive investigations into the nature of sewer sediment were carried out for WRc (and Tayside R. C.), starting in 1987. The report entitled 'Sediment Behaviour in Combined Sewers' was to form part of an important database for use within the Urban Pollution Management programme, and also to contribute to the development of the MOSQITO sewer flow quality model. The main aims of this study were:

- Identification of the nature of sediments;
- Monitoring of sediment movement;
- Assessment of provenance of the sediments;
- Differentiation between sediment transport mechanisms;
- As far as practicable, to relate sediment transport to hydraulic performance;
- Quantification of any in-sewer changes in the physical and chemical composition of sediments during transport and in-pipe storage.

These investigations were focused on the main Dundee interceptor sewer (at the head and foot of the Murraygate) and the upper Perth Road catchment. The programme achieved some of the original objectives, but initially could not provide sufficient information regarding the provenance of sediments to produce meaningful results. However the investigations made during the programme served to act as a springboard for future studies of sediment provenance and movement.

The Movement of Cohesive Sediment in a Large Combined Sewer - David J.J. Wotherspoon (1994)

Historically, much of the work carried out in the area of sediment transport has been concerned with the analysis of non-cohesive sediment particles. The Wotherspoon study was a joint study with Swansea University, and concentrated specifically on the behaviour of cohesive sediments with changes in hydraulic conditions, sediment bed deposit depth and suspended solids flux. Detailed investigations focused on the Dundee Interceptor Sewer due to the wide knowledge base which had already been developed in this location. A strong emphasis was placed upon the rheology of bed deposits, leading to the development of a novel erosion prediction model. Further details of this study are provided in Chapter 2.

Solids Transport in Combined Sewerage Systems - Brian P. Coghlan (1995)

The principal aim of the Coghlan study was the development of a method by which rates of transport and characteristics of suspended and bed load material could be assessed. The original studies concentrated on the Dundee Interceptor Sewer, with verification work of the original findings carried out in the Upper Perth Road area of Dundee. Site specific models for suspended load transport were developed, with alterations made to produce non-site specific relationships.

Near Bed Solids Transport in Combined Sewers - Scott Arthur (1996)

The Arthur study took the form of a collaborative research project involving The University of Newcastle upon Tyne, the University of Sheffield and Tayside Regional Council Department of Water Services. Existing methods of near-bed sediment transport prediction methods were evaluated, and a new relationship proposed. The project focus was based around material moving near the bed, and examines the nature of this material. A link between pollutants observed during first foul flush phenomena and pollutants associated with the material in transport at the bed is also proposed. Studies were carried out at the Dundee Interceptor Sewer (head); Dens Brae Sewer (foot); and Constable Street Sewer (some 1.7 km along the Interceptor from its head). Further details of these investigations are provided in Chapter 2.

3.2.2.2 Dundee Interceptor Sewer

The Dundee Interceptor Sewer has been the principal focus of many sewer sediment studies in Dundee (including those detailed above). This wealth of knowledge was used as a foundation for further investigation within this study.

The Dundee Interceptor Sewer accepts flows from the western half of the Central area catchment, with significant inputs from the City Centre, Marketgait, Hawkhill, Polepark, Blackness and East Lochee areas of the city. The interceptor sewer runs from the City Centre at the junction of Reform Street and High Street, along the commercial area of Murraygate and then through Cowgate to the east of the city centre (Figure 3.5). Historically (and during this study), from this point the sewer continued along Constable Street and then Blackscroft before turning South near Roodyards Road to a short sea outfall. Currently this outfall has been replaced with an outlet to the King George the V Pumping Station as part of the Tay Estuary PPP scheme. The length of sewer monitored for the study reported here stretches from the head of the interceptor sewer through High Street and into Murraygate. The total length of this section is 150 m. A more detailed plan of this area is shown in Figure 3.11. The invert level throughout this length was measured in detail and is shown in Figure 3.7.

The chamber from which the interceptor sewer originates is significant to the operation of the sewer length, as it contains both a flow diversion gate and a large silt trap. During the study reported here and a number of previous studies, this silt trap has been covered over with a length of prosthetic sewer to allow measurements of near bed sediment transport rates to be made.

The cross-section of the sewer throughout this length is egg shaped, with dimensions varying from 1210 mm X 810 mm at its start, to 1780 mm X 1625 mm at Blackscroft. This cross section was constructed in the 1880's from brick and is in relatively good condition in a core area of the city centre. A sample of the sewer's cross-section is shown in Figure 3.9

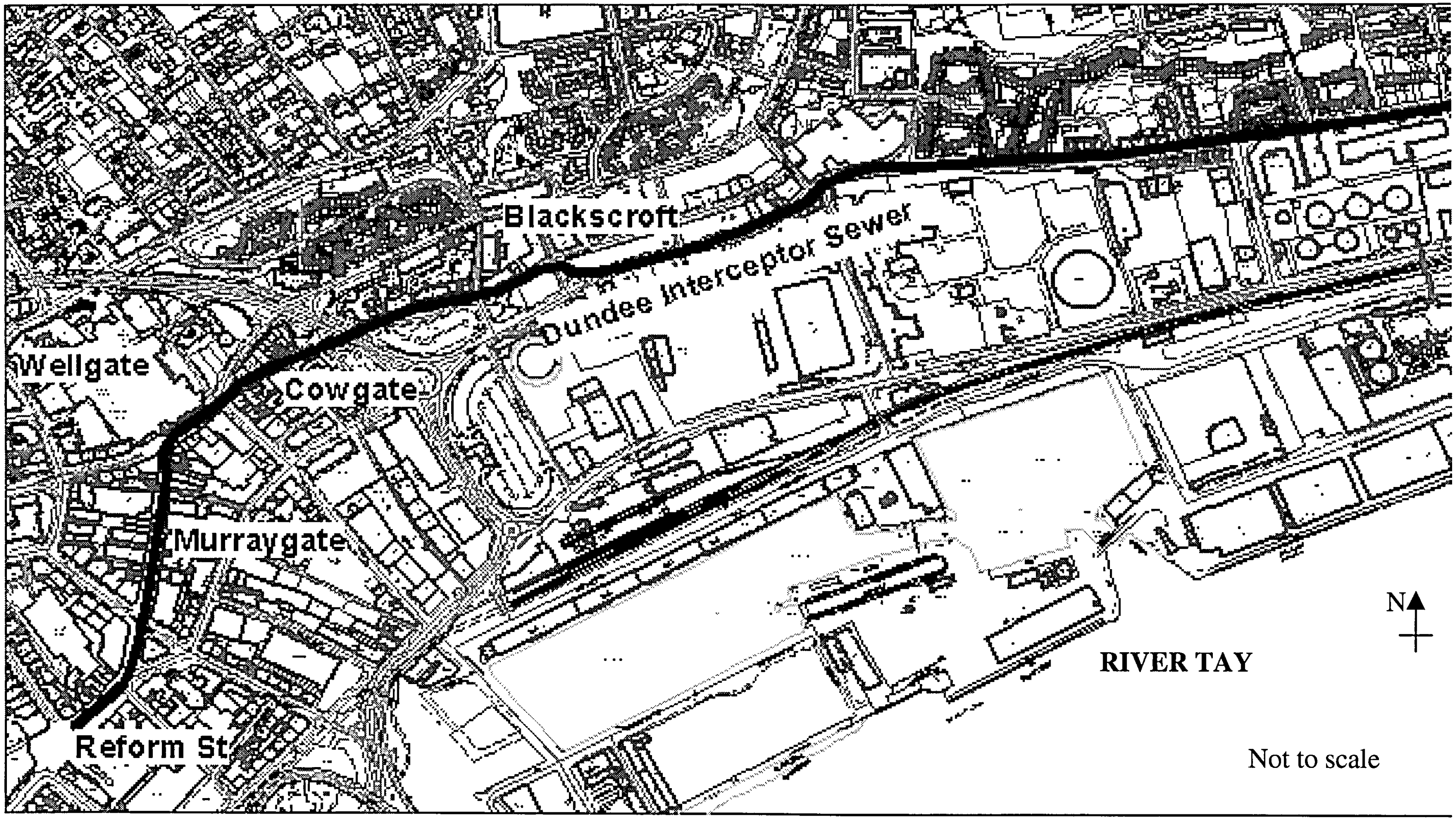


Figure 3.5 - Main Dundee Interceptor Sewer

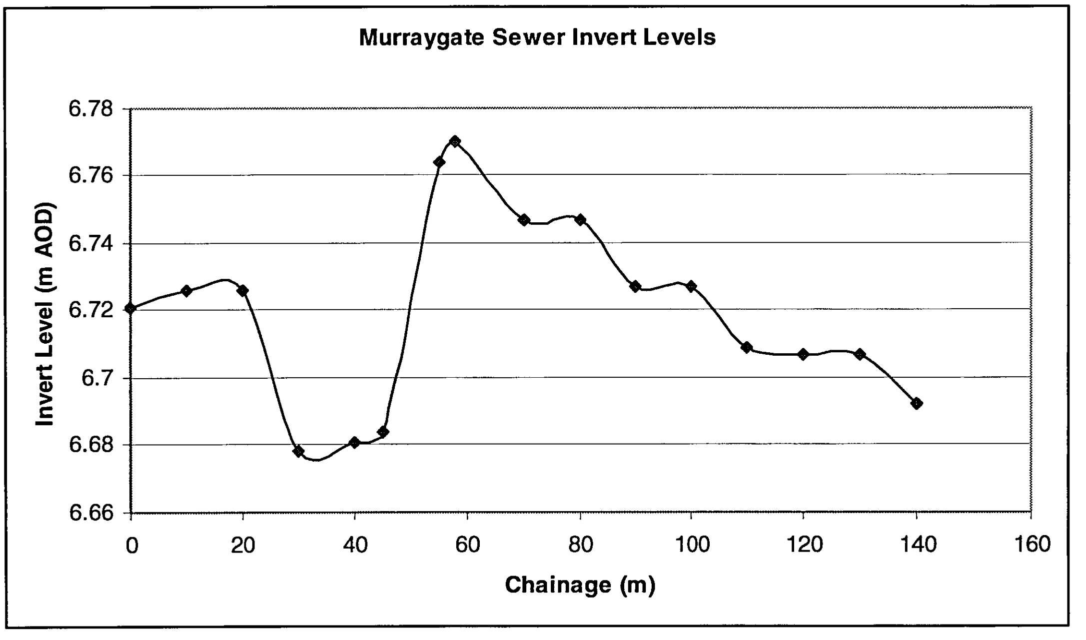


Figure 3.7 - Murraygate sewer invert levels

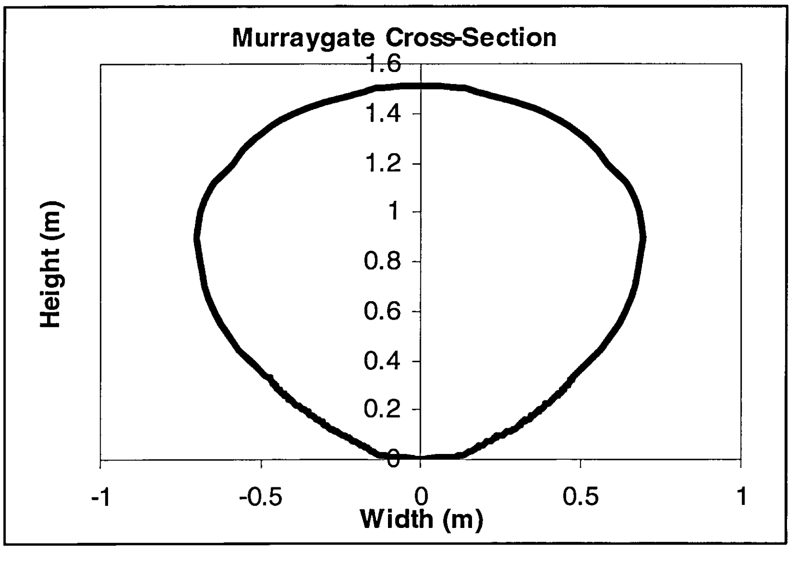


Figure 3.9 - Murraygate sewer cross-section

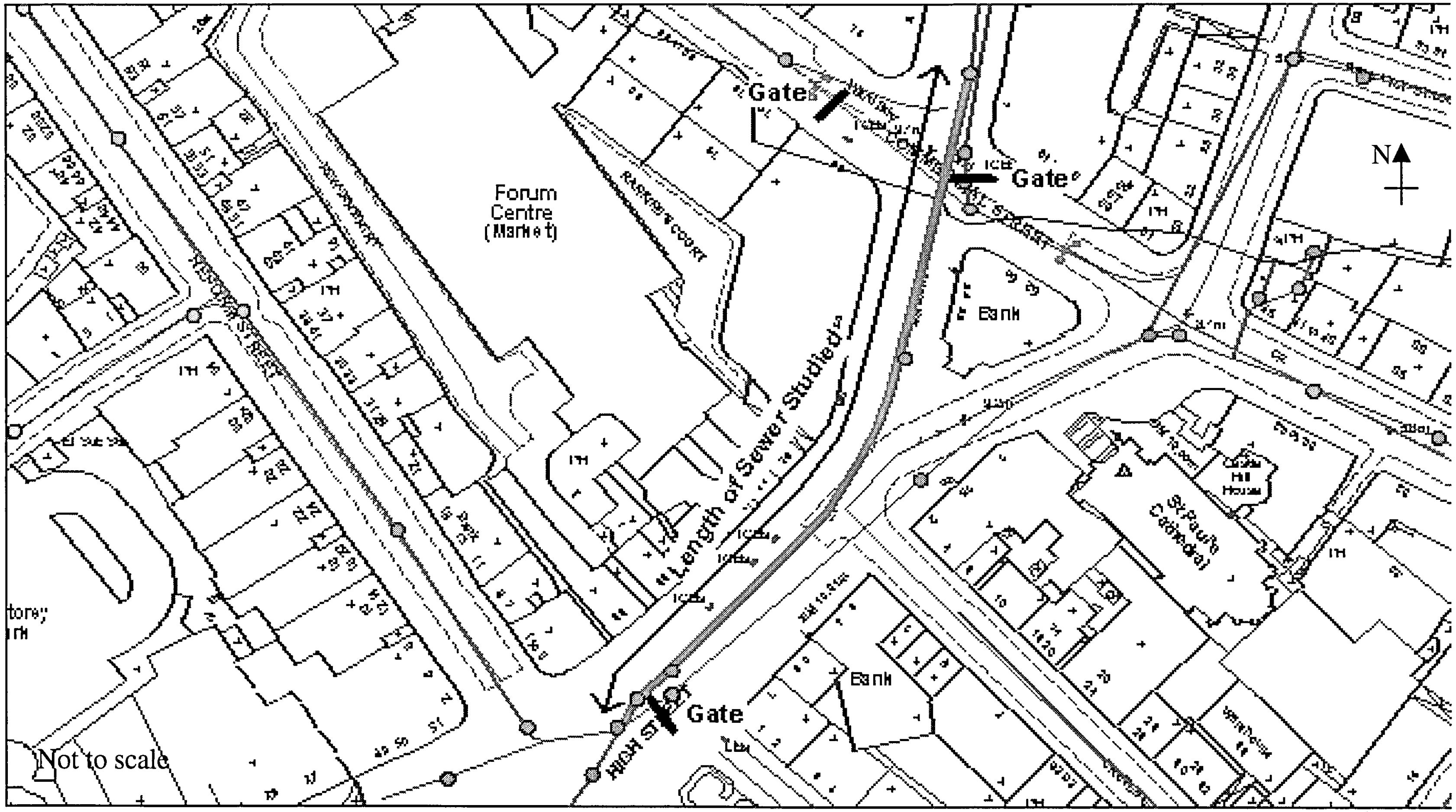


Figure 3.11 - Length of Interceptor Sewer Studied

As a result of the history of investigations undertaken in the upper reaches of the interceptor sewer, a number of structures exist within the sewer to assist in the measurement of sediment behaviour and characteristics.

3.2.2.3 Drainage Sediments in the City Centre Catchment

A combination of the significant sediment studies carried out within the Dundee Central catchment and a wealth of hydraulic modelling experience have led to a broad knowledge base of the locations of sediment related problems within the catchment. These knowledge bases were combined with up-to-date man entry and closed circuit television (CCTV) surveys to provide a time history of sediment deposition throughout the city.

The most extensive study examining Dundee sediments was the WRc investigation carried out in 1989. This detailed study concentrated predominantly on the Dundee interceptor sewer, and Perth Road subcatchment. However, a city-wide survey was undertaken to locate deposits. Figure 3.13 shows where sediment deposits were found in Dundee's City Centre during the 1989 survey (highlighted in red). Flow patterns in the system have not been altered significantly since this survey took place.

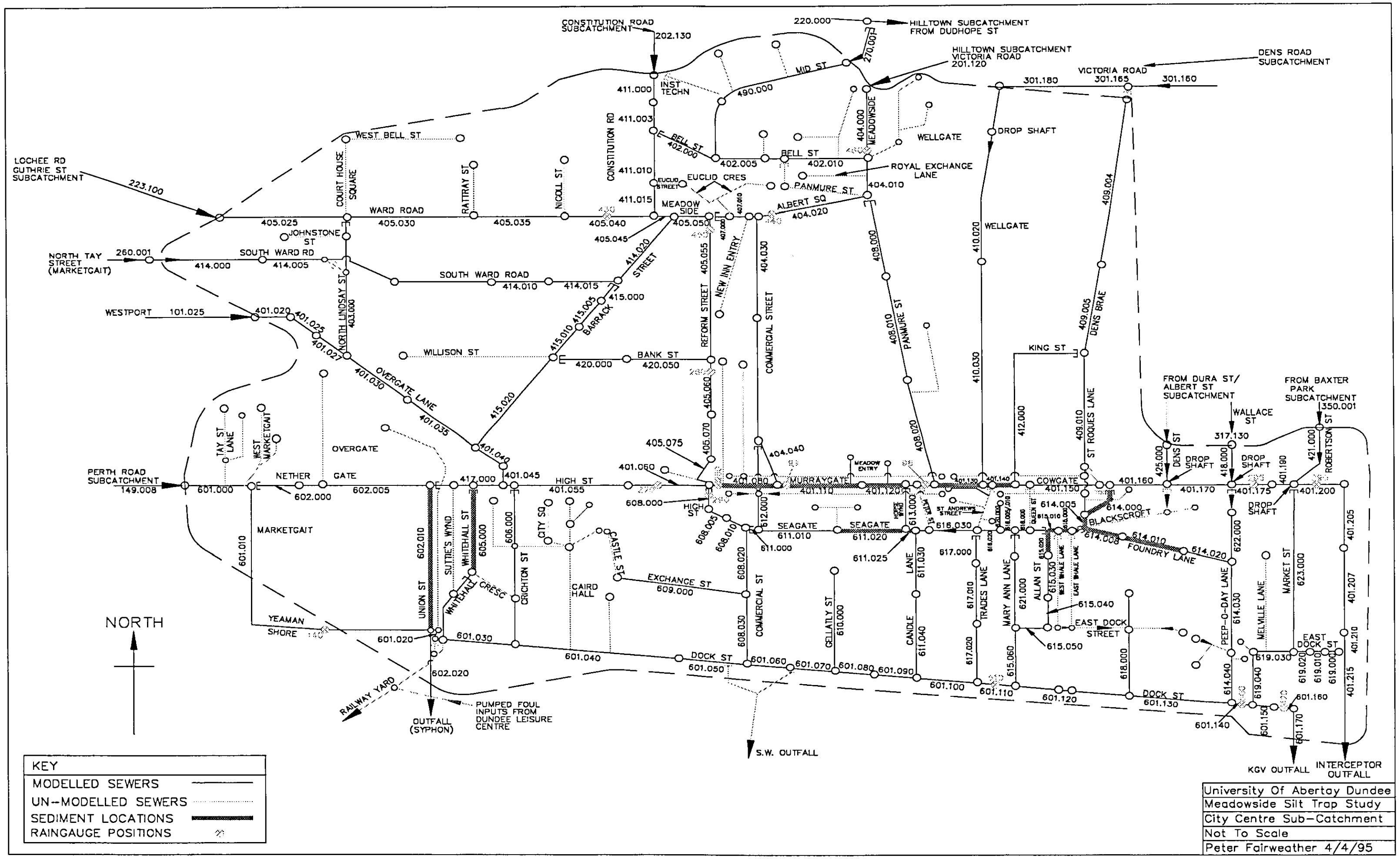


Figure 3.13 – 1989 survey - Schematic layout of sediment location in central Dundee

The 1989 survey was used in conjunction with operational experience to produce a register of sediment deposit locations to be used in the Dundee HydroWorks flow model. This data-set can be considered the most accurate long term assessment of Dundee's sewer deposits. These locations are summarised below in Table 3.2.

CATCHMENT	PIPE No.	STREET	SEDIMENT DEPTH (mm)
CITY CENTRE	219_010.1	Caldrun Street	50
(DCASM 8/1)	401_045.1	High Street	100
SUB-MODEL: CITY CENTRE	401_055.1	High Street	100
	401_060.1	High Street	100
	401_080.1	High St./ Murraygate	100
	401_110.1	Murraygate	100
	401_120.1	Murraygate	200
	401_130.1	Cowgate	200
	401_140.1	Cowgate	200
	401_150.1	Cowgate	100
	401_160.1	Cowgate	100
	401_170.1	Cowgate	100
	401_175.1	Blackscroft	100
	401_190.1	Broughty Ferry Road	100
	408_010.1	Panmure Street	100
	408_020.1	Panmure Street	100
	417_000.1	High Street	100
	421_000.1	Broughty Ferry Road	150
	421_010.1	Broughty Ferry Road	100
	425_050.1	Constable Street	50
	601_040.1	Dock Street	50
	601_050.1	Dock Street	50
	601_060.1	Dock Street	50
	601_070.1	Dock Street	50
	601_080.1	Dock Street	150
	601_090.1	Dock Street	150
	601_100.1	Dock Street	50
	601_110.1	Dock Street	200
	601_120.1	Dock Street	50
	601_130.1	Dock Street	50
	601_140.1	Dock Street	50
	601_150.1	Dock Street	50
	601_160.1	Dock Street	50
	601_170.1	Dock Street	50
	602_005.1	Nethergate	100
	602_010.1	Union Street	100
	605_000.1	Whitehall Street	100
	608_005.1	High Street	100
	611_020.1	Seagate	100
	614_000.1	Blackscroft	100
	614_005.1	Blackscroft	100
	614_008.1	Foundary Lane	50
	614_010.1	Foundary Lane	50
	614_020.1	Foundary Lane	100
	615_010.1	Seagate	150
	615_010.2	East Whale Lane	50
	615_011.1	East Whale Lane	150
	615_020.1	Allan Street	50
	615_050.1	East Dock Street	250
	615_050.2	East Dock Street	125

	615_060.1	East Dock Street	300
	616_010.1	Seagate	200
	616_015.1	Seagate	200
	616_020.1	Seagate	50
	617_010.1	Trades Lane	100
	617_020.1	Trades Lane	100
	621_000.1	Mary Ann Lane	100
POLEPARK & LOCHEE RD. (DCASM 3/1) SUB-MODEL: SUBA	224_000.1	Milnes Wynd	100
DENS ROAD (DCASM) SUB-MODEL: SUBB	301_150.1	Dens Road	100
	306_000.1	Law Road	100
	306_010.1	Leng Street	100
	313_010.1	Main Street	75
	314_040.1	Arkley Street	75
	317_010.1	Clepington Road	25
	317_020.1	Clepington Road	50
	319_000.1	Molison St. / Eliza St.	75
	323_010.1	Craigie Street	50
	327_000.1	Lyon Street	50
	328_010.1	Arbroath Road	100
	328_020.1	Arbroath Road	100
	330_070.1	Albert Street	50
	334_010.1	Pitkerro Road	75
	336_000.1	Dundonald Street	50
	337_000.1	Glamis Street	50
	337_000A	St. Salvador Street	25
PERTH ROAD UPPER (DCASM 1/1) SUB-MODEL: SUBA BLACKNESS/LOWER PERTH RD. (DCASM 2/1) SUB-MODEL: SUBA	122_002.1	Perth Road	50
	123_000.1	Perth Road	50
	123_001.1	Perth Road	50
	112_030.1	Blackness Road	50
	112_040.1	Blackness Road	50
	114_040.1	Hawkhill	50
	119_000.1	Hunter Street	50
	123_016.1	Perth Road	100
	123_018.1	Magdalen Yard Road	50
	142_001.1	Perth Road	100
	142_002.1	Perth Road	100
	142_003.1	Perth Road	100
	142_004.1	Perth Road	100
	142_006.1	Perth Road	150
	143_000.1	Blackness Road	50
	149_000.1	Perth Road	150

Table 3.4 - DCASM sediment locations

These data are also shown diagrammatically on a plan of the Dundee Central Area Sewer Model (DCASM) in Figure 3.15.

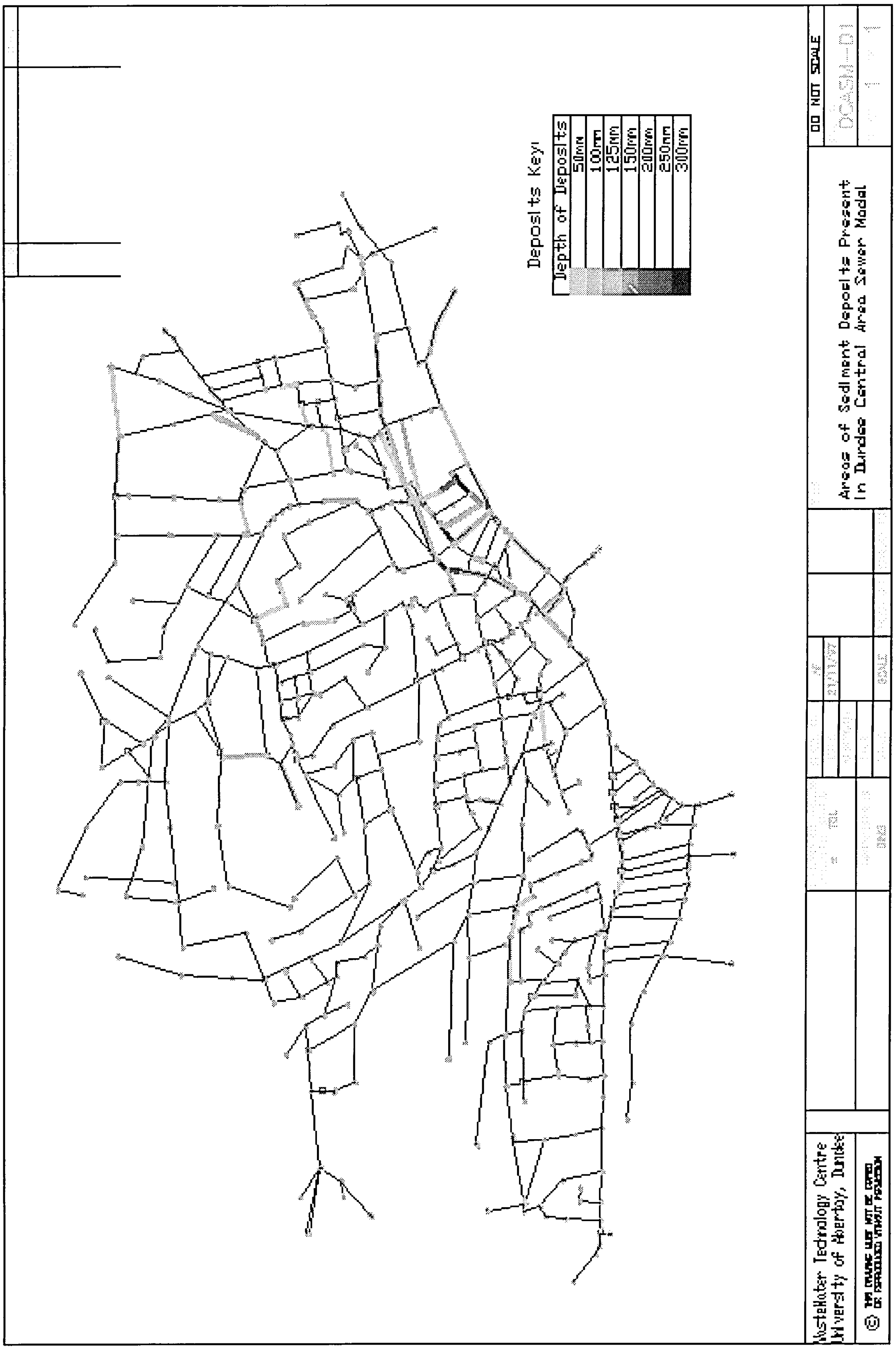


Figure 3.15 - Sediment Locations Used In DCASM

The pipe lengths, which have been shown to be most commonly affected in the City Centre area may be summarised as:

- Interceptor sewer from Murraygate to Cowgate;
- East Seagate;
- Foundry Lane;
- Union Street;
- Whitehall Street;
- Dock Street system (N.B. Details not shown on plan, exact locations not known)

3.2.2.4 Characteristics of Dundee Sediments

The Dundee interceptor sewer is in excess of 1500 mm high along most of its length, and as a result, man entry inspection is therefore possible allowing widespread sampling to take place. Deposits along this length are common and are subject to frequent removal. Prior to cleaning taking place, pipe deposits of up to 500 mm have been recorded (Ashley, 1993). Most of the sediment deposits in the City Centre sub-catchment are mixed (WRc) class A/C (Table 2.6) in nature and are generally continuous in the interceptor sewer, with a depth of 50-250 mm.

3.2.2.4.1 Particle sizes

As part of the Wotherspoon study (1994), 55 samples of sediment bed deposit were taken from the interceptor sewer. These samples were then oven dried and dry sieved down to a minimum sieve size of 63 μm. The results from these samples yielded a range of particle size distributions (see Figure 3.17 and Table 3.5), with a d_{50} of 417 μm.

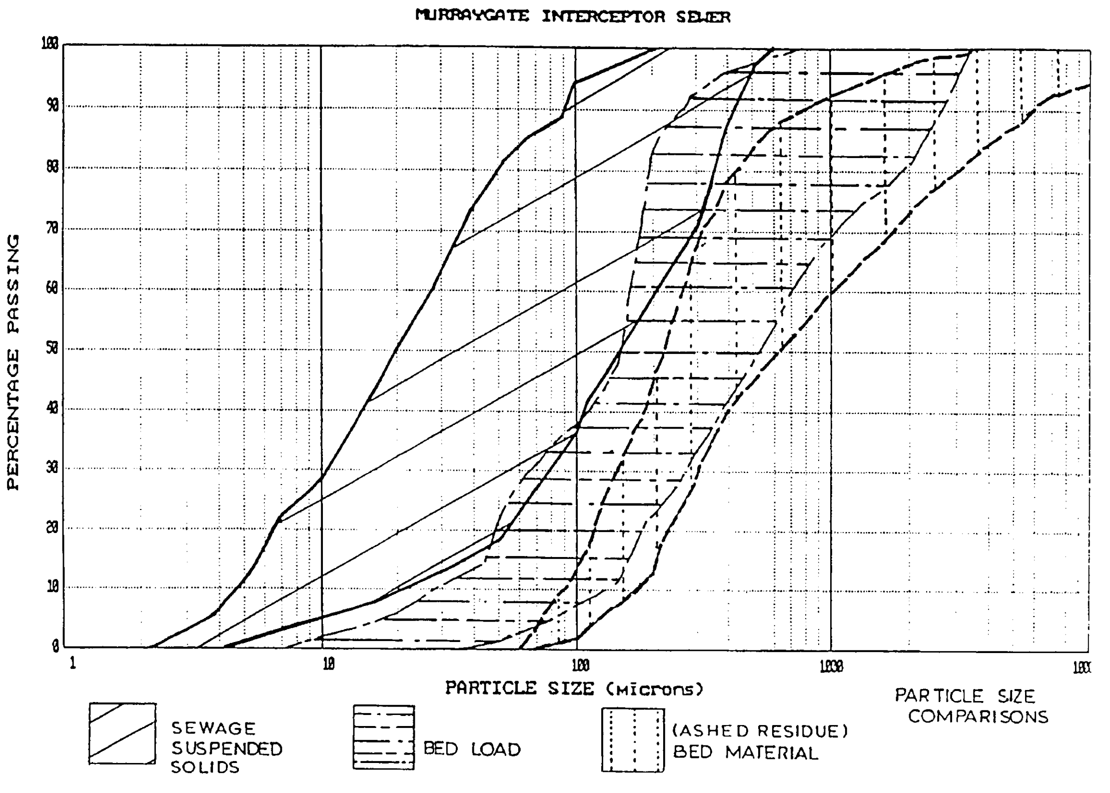


Figure 3.17 - Dundee Interceptor Particle Size Distributions

Fraction	Lower Limit	Upper Limit	Average
d_{10} (μm)	86	179	132
d_{35} (μm)	171	352	262
d_{50} (μm)	216	619	417
d_{75} (μm)	363	2244	1303
d_{90} (μm)	765	5724	3244

Table 3.5-Dundee Interceptor Sewer: Range of Particle Sizes

Studies of bed deposits have also been carried out in the Perth Road area of Dundee. As part of the WRc investigation (1993), some 20 samples were taken from three separate locations on the main Perth Road sewer. The resulting sieve analysis produced particle size envelopes for each site. By combining the three sites, an envelope for the entire sewer length can be developed (see Figure 3.19).

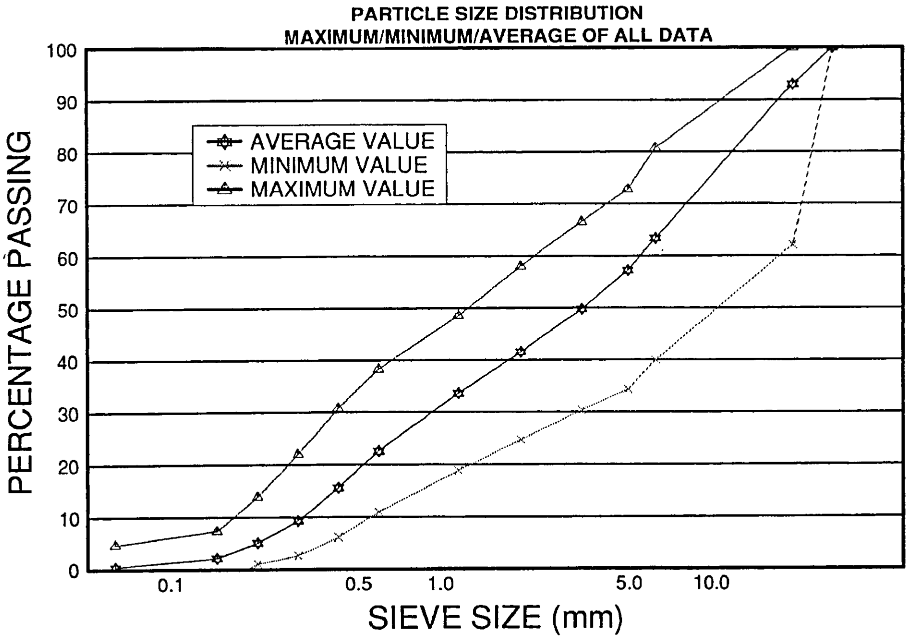


Figure 3.19 - Particle Size Envelopes: Perth Road

d_{10} (mm)	d_{50} (mm)	d_{90} (mm)	% Volatiles
0.2 - 0.5	1.2 - 10.0	10.0 - 24.0	0.7 - 10.4

Table 3.6 - Perth Road Sewer: Range of Particle Sizes

3.2.2.4.2 Bulk Density

The bulk density of a sample is defined as a sample's total mass divided by its volume and is a parameter widely used in the estimation of erosion. The bulk density of any particular sample will be determined by the characteristics of the sediment contributing to the deposit (density, particle grading, and moisture content) and also the effects of sample consolidation with time. Wotherspoon (1994) attempted to correlate bulk density with other sediment parameters (e.g. yield stress, water content), but found only limited correlation. Within the Wotherspoon study, Dundee interceptor sewer deposits were found to have bulk densities ranging generally from around 1400 kg/m^3 to 1800 kg/m^3, with a mean of around 1580 kg/m^3 (S.D.~230 kg/m^3). Samples taken from the Perth Road sewer during the WRc study (1993), showed the deposits to be of a higher bulk density, with a mean of 1807 kg/m^3. These samples were more granular.

3.2.2.4.3 Settling velocity

Particle settling velocities are difficult to determine accurately due to particle aggregation and the inherent bias of any test method used, but range from very low colloidal suspensions with $w_s < 0.01$ mm/s to particles which settle too fast to measure in standard apparatus. In Dundee, tests have been carried out to estimate the settling properties of moving sediments in the Dundee Interceptor Sewer and Perth Road trunk sewer. Wotherspoon (1994) measured the settling characteristics of suspended sediments in the field using the Owen Tube and SDD methods. The tests performed by Wotherspoon suggested that the Owen Tube apparatus gave the best indications of sediment behaviour in the field. Table 3.7 shows results for these Owen tests on the Dundee samples. Further details of the various settling tests are provided in Appendix C.

Date	**Flow Depth (m)**	**Flow Velocity (m/s)**	**Conc. (mg/l)**	**w_{50} (mm/s)**	**w_{75} (mm/s)**	**w_{75}/w_{50} (mm/s)**
24/4/90	0.33	0.28	178	<0.0010	0.0457	>46
25/4/90	0.33	0.29	225	0.0062	0.3860	62.3
25/4/90	0.31	0.26	226	0.0076	1.0926	143.8
26/4/90	0.33	0.30	208	0.0130	0.2384	18.3
30/5/90	0.32	0.33	224	0.2333	2.6578	11.4
31/5/90	0.33	0.31	263	0.0219	1.3263	60.6
5/6/90			281	0.1385	1.4785	10.7
6/6/90			190	<0.0010	0.5000	>500
30/5/90			276	0.0038	3.9295	1034.1
31/5/90			225	0.0071	0.4380	61.7
31/5/90			237	0.0059	0.3930	66.6

Table 3.7 - Dundee Sewage Settling Characteristics (from Wotherspoon, 1994)

The characteristics of the material moving near the bed were examined in great detail as part of the Arthur study (1996). Arthur attempted to take 'undisturbed' samples of near bed solids and measure (among other parameters) the settling velocities of the material using the UFT apparatus. As a result of the difficulties of obtaining undisturbed samples, only limited testing was carried out.

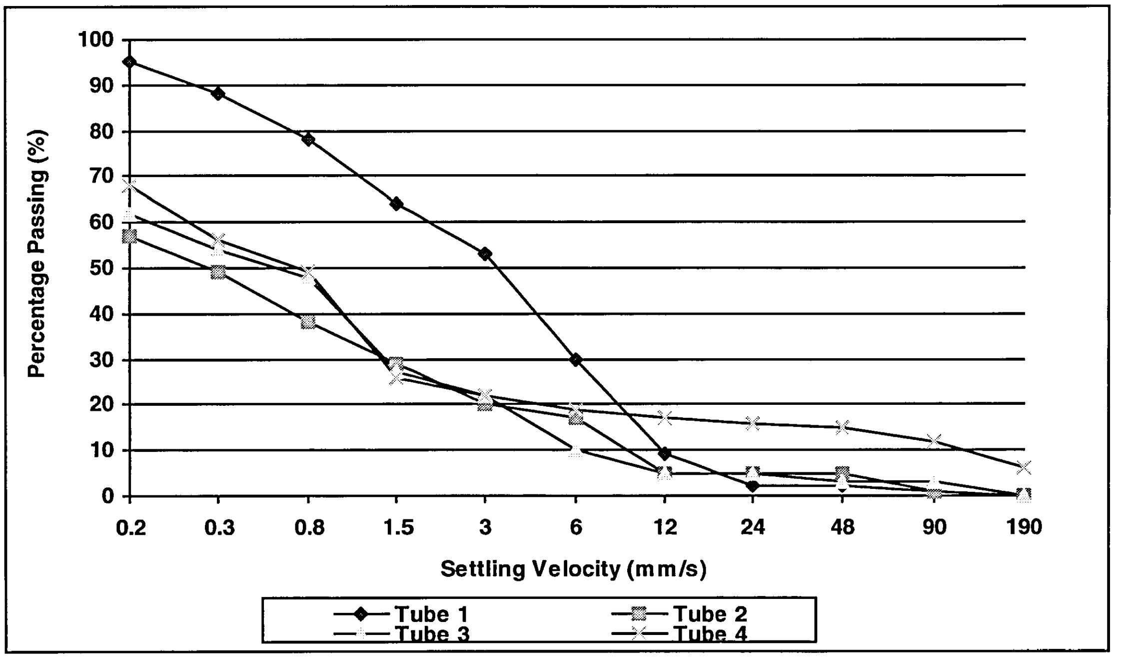


Figure 3.21 - Settling Velocity Data: Dundee Interceptor

Four sampling tubes at varying depths above the bed (5mm, 150mm, 300mm and 450mm) were inserted into the flow and sewage samples extracted. Figure 3.21 shows that the material moving nearest the bed (Tube 1) contained particles of the highest settling velocity. The Dundee data generally give lower settling velocities than those measured in equivalent French studies (Arthur, 1996), although this is believed to be a result of the non-selective nature of the French gully pots.

3.2.2.4.4 Resistance to Erosion

It is clear that sediments exhibit an *apparent* yield stress when subjected to controlled rates of applied stress. Sewer sediments have been observed to be non-Newtonian visco-elastic materials, and to have widely differing critical yield stresses ($5 < \tau_y < 2500$ N/m^2) measured for the Dundee interceptor deposits. The differences observed may be attributed to the post-depositional history of the deposits and the relative importance of the organic material. The measured yield stress can be considered as corresponding to the fluidisation of the material, and hence a measure of the onset of erosion. The *apparent* yield strength was found to be most closely related to the liquid content of the sample, as given by (Wotherspoon and Ashley, 1992):

$$\tau_y = 9.66x10^7 m^{-3.1682} \qquad \text{N/m}^2 \qquad \textbf{Equation 3-1}$$

3.2.2.4.5 Pollutant Potential

In general terms, it can be said that the most polluting sediments are also those most prone to erosion. Table 3.8 given below shows typical pollutant levels found in Interceptor Sewer samples of sewage, bed deposits and near bed solids. It must be stressed however, that these are highly variable both spatially and temporally. The most concentrated are the materials moving along the sewer just at the bed (along a clean sewer, or travelling over a deposited bed). The figures in Table 3.8 are a summary of the findings of Wotherspoon (1994), Arthur (1996) and this study.

Sediment	Bulk density (kg/m^3)	Volatile solids (%)	COD (g/l)	BOD (g/l)	AmmN (g/l)	Specific gravity
Near-bed solids	<1000-1518	0.5-84	85-328	1-685	0.068-2.3	<1-1.45
Bed deposit	1510-2095	<20	0.1-108	0.02-13.9	0.01-1.9	2.65
Sewage	1000	<95	0.346	0.143	0.005	1.0-2.65

Table 3.8 - Polluting Characteristics of Dundee Interceptor Sewer Sediments

3.2.2.4.6 Bacteria

The examination of bacteria populations in sewer sediments has so far concentrated largely on populations of faecal coliforms, total coliforms, and faecal streptococci as indicators of potential presence of pathogens and viruses. Whilst a number of studies have examined sewage quality, few have considered sediments. The studies undertaken in Dundee to assist with the development of guidelines for the control of sewage discharges into coastal areas and the inclusion of bacteria in the MOUSETRAP model have also been principally concerned with sewage, but have looked for evidence of sediment erosion adding to bacteria numbers. The sediment data for bed deposits are summarised (Ashley & Dabrowski, 1995) and compared with sewage data, in Table 3.9.

Bacteria Numbers (million/100 ml)					
		DWF		***Storms***	
	Sediment*	***Summer***	***Winter***	***Summer***	***Winter***
TC	0.9 - 2631	20.5	6.27	3.8 - 113	2.2 - 59
FC	0.01 - 314	2.6	-	0.3 - 27	0.1 - 8.5
FS	< 0.1 - 39	0.14	0.28	0.03 - 1.25	0.04 - 2.0

** Lower counts were observed for winter samples*

Table 3.9 - Bacteria Numbers in Sediment and Sewage in Dundee

The results clearly show that sewer sediments are bacteria accumulators/nurseries. More recent results from Dundee (McGregor et al, 1995) investigated the sensitivity of the indicator species to test procedure protocols. The results presented above were obtained using the standard blending protocol preparation for samples, whereas the later tests used protocol options in order to apportion bacteria to the relative solid-liquid phases. The results showed that the blending protocol, assumed to provide a test sample which will yield extreme values for physical and chemical testing, may well be compromising the viability of the micro-organisms. Thus the earlier results (Table 3.9) may give an underestimate of sediment bacteria numbers.

3.2.3 Dighty Catchment

The Dighty catchment is the largest of the three main Dundee catchments (960.3 ha) and drains the North and East parts of the city, the area also serves the largest population at around 53,284. The catchment is predominantly combined, with a limited number of separate drainage systems in some of the newer industrial areas and housing estates. The network of pipes is dendritic and is centred around a spinal trunk sewer running through the Dighty valley, to the Panmurefield Screening Chambers. Table 3.10 (below) shows a summary of surface details for the catchment.

Total Catchment Area	994.16 ha
Total Hydrological Catchment	493.21 ha
Total Paved Area	178.76 ha
Total Roof Area	127.39 ha
Total Permeable Area	187.05 ha

Table 3.10- Permeability Survey (Source: - Dighty Catchment Model Development Report, Montgomery Watson, 1995)

Four pumping stations are employed to convey flows to the treatment facilities at a maximum rate of 181 l/s, any excess flows are discharged either from combined sewer overflows (in the case of the three smaller pumping stations) or via a short estuary outfall.

One sediment trap is located at the head of the system and has been used in these investigations.

3.2.4 Invergowrie Riverside Drainage Scheme (IRDS)

The Invergowrie-Riverside Drainage System (IRDS) is a self-contained catchment located to the North-west of Dundee's City Centre. The drainage system is predominantly combined, and serves a population of around 25,000. A large proportion of this population is located in the North-east of the catchment in the subcatchments of Lochee, Buttar's Loan, Menzieshill and Charleston. Flows from this area are then conveyed under high gradients to a relatively flat interceptor sewer which carries flows from a pumping station along the coastline to treatment facilities. The system has three principal trunk sewers:

- Kingsway trunk sewer serving Lochee, Charleston & Invergowrie;
- Menzieshill trunk sewer serving Menzieshill;
- Ninewells trunk sewer serving Ninewells hospital and commercial development areas.

The total area served by the system is 370 ha, of which 222 ha have been estimated as being impermeable. The land uses of the catchment are typically domestic and commercial with limited industrial inputs present. Treatment facilities located in the catchment consist of a large screening plant intercepting all particles larger than 12 mm.

No consistent studies have been undertaken of the sedimentation in this catchment. However, sedimentation and its impacts have been experienced at the screening facilities. A rainfall event on 11/6/97 resulted in 7810 kg of sediment arriving at the IRDS screening plant in an 8-hour period. This resulted in damage to the screening chamber screens. The costs associated with the screen replacement, down-time and maintenance man-hours amounted to approximately £10,000.

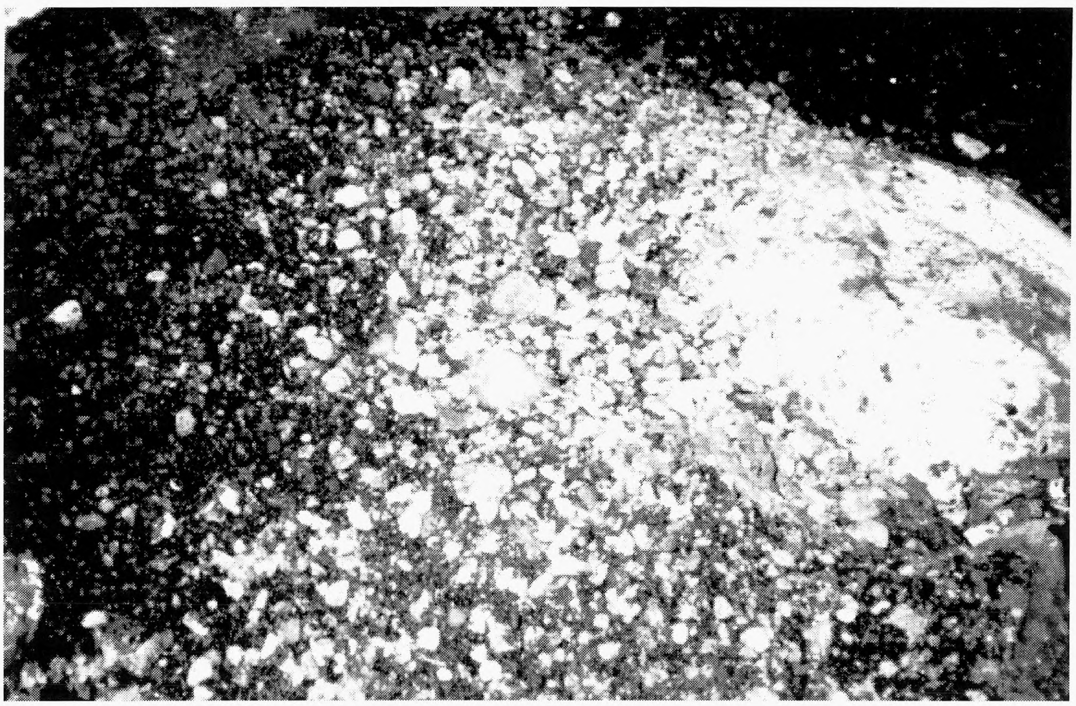


Figure 3.23 - Sediments removed from IRDS screens

Figure 3.25 - Large particles present in sample

Figure 3.23 and Figure 3.25 show examples of the types of sediment impacting the screens. These can be seen to vary from small grits and gravels to large half-bricks and construction debris. The condition of the individual particles indicated that a large proportion of them had been present within flowing water for a significant time. The findings during this investigation further highlight the significance of drainage sediments in the operation and maintenance of sewerage networks.

3.3 Forfar Drainage Network

Forfar is situated in Eastern Scotland, approximately 21km north of the city of Dundee and is the main town in the county of Angus. The population of the town is 12,961 (General Register for Scotland, 1997). The town of Forfar originated from a small market area and developed in a radial pattern from the central High Street during the 19th century. Industry in the area is mainly limited to Orchardloan Trading Estate and the Strathmore Mineral Water Company facilities.

The burgh of Forfar lies within the catchment of Forfar Loch and as a consequence all drainage from the town and its outlying areas discharges to this waterbody. This includes all surface runoff as well as discharges for which the local water authority has a responsibility. The loch is located on the west side of the town and is fed by two culverted Burns (Treacle Burn and Hornie Burn), originating in the surrounding hills. The natural drainage of the catchment generally runs from east to west. The Loch itself is 1.5km long and 9 metres deep and discharges into Dean Water, which flows via Glamis to the River Isla and the sea.

Forfar is served by a sewerage network which has developed in a number of directions as the town has grown. The southern catchment is served by a combination of separate, combined and dual manhole systems, which drain to the Wastewater Treatment Works near Forfar Loch (the treatment plant discharges to the loch). The northern catchment consists of separate and combined systems draining to the Queenswell Road Pumping Station, which transfers flows to the Wastewater Treatment Works (WWTW). The principal trunk sewers serving the town transfer flows from the centre of town, along the bank of the loch, to the treatment plant via parallel 900mm and 600mm diameter concrete sewers. The network contains overflows that drain excess flow to Forfar Loch via the culverted watercourses. This is of particular relevance in the case of the trunk sewers as SEPA require the full capacity of both pipes to be kept at all times in order to prevent unnecessary discharges. These trunk sewers are of slack gradients and are observed to suffer from severe sedimentation problems.

Forfar Wastewater Treatment Works was originally constructed in 1953 and consisted of a settlement tank, filter bed and a humus tank. As the town expanded, the council provided separate drainage systems to reduce the load on the plant. In 1967 a substantial section of the trunk sewer was replaced. In the early 1970's an activated sludge process was introduced and secondary settlement tanks were added. By the late 1980's the plant could not sustain any further domestic and industrial inputs, which restricted development in the burgh. The plant was viewed to be complex and ageing and only met consent standards 60% of the time. There was also local concern over algal blooms in the Loch and in the Dean Water. In 1990 a WRC report, concluded that a very low flushing rate and high nutrient inputs from the catchment caused this eutrophication. It was discovered that significant phosphorus and nitrogen reserves had built up in the sediment in the Loch. In 1994, under the Urban Waste Water Treatment (Scotland) regulations, Forfar Loch was designated 'sensitive'. Babcock Water Engineering Ltd was appointed in 1997 to construct a £5.2 million new treatment works, to address some of these problems.

In 1996, NoSWA carried out an investigation to determine the improvements that were required for the Forfar sewer system. The outcome was a phased improvement strategy to be implemented in three stages. Phase 1 involved the removal of two overflows, the upgrading of one CSO, a new pumping station and storage facility at Queenswell Road, and importantly, the construction of two new silt traps on the two principal trunk sewers. Phase 2 of the improvements saw the construction of a new sewer along Queenswell Road. The third phase will involve the construction of two storage tanks at Myre Road and Academy Street. Phases 1 and 2 have been completed, while phase 3 is proposed for 2004-5.

The improvements carried out to date have facilitated hydraulic improvements at a number of locations throughout the town. However, the high sedimentation rates of the trunk sewers have persisted. This has led to the rapid filling of the recently constructed silt traps to the extent that they have a negligible impact on the reduction of in-pipe silting. The new treatment works has also suffered from flush loadings believed to be associated with the sediments located in the trunk sewers. These problems were of sufficient magnitude to force Scottish Water (previously North of Scotland Water) to contact the sewer sediment research team at the University of Abertay Dundee, in order to request a study of the mechanisms of the deposits and a suggestion of a "best" method of sediment management. The findings of this work are discussed further in the following sections and in Chapter 4.

3.4 *Sediment Trap Details*

Three separate sites were chosen for detailed field investigation for the sediment and trap performance studies. The sites were chosen as the traps at these locations are subject to very different flow and sediment loadings, allowing the trap performances to be assessed over the widest possible conditions. The following sections provide details of each of the sites chosen.

3.4.1 Constable Street Silt Trap

The Constable Street chamber is located on the Dundee Interceptor Sewer on exit from the City Centre.

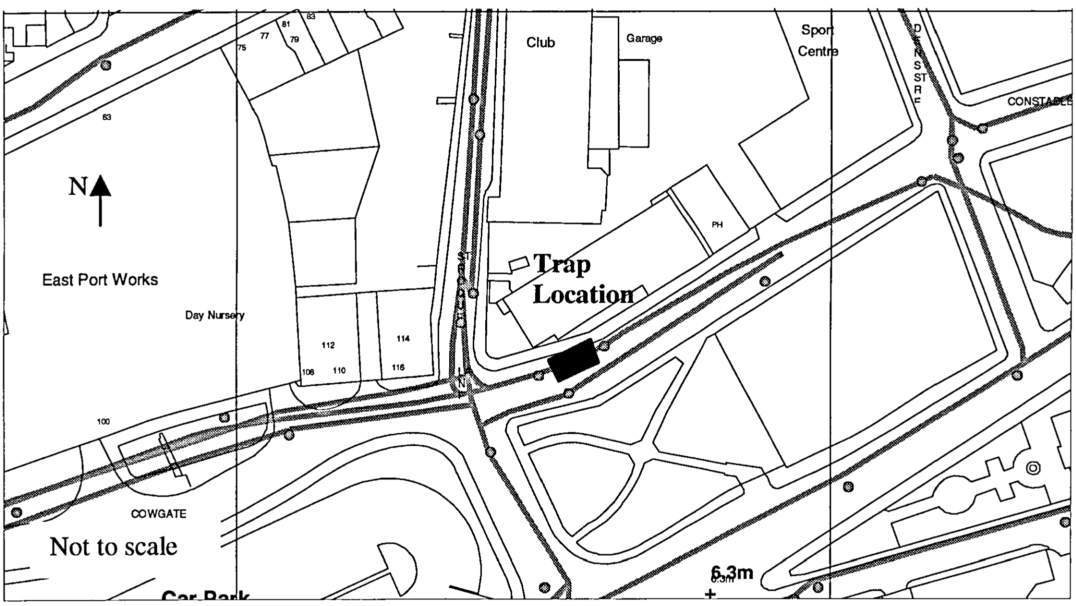


Figure 3.27 - Constable Street trap location

The sewer at this point is approximately egg-shaped in section, of height 1780 mm and width 1625 mm. The pipe gradient along this length of sewer is approximately 1:1750 (0.057%). Figure 3.29 shows the profile of the pipe upstream of the chamber, the chamber is located at chainage 700 m.

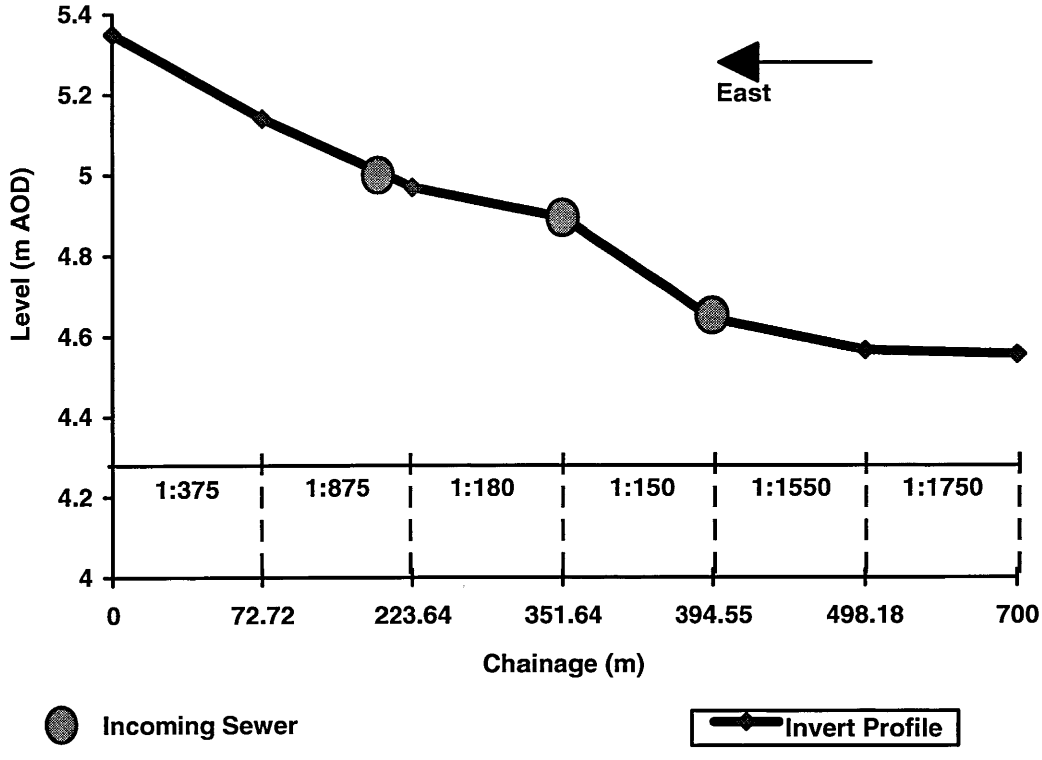


Figure 3.29 - Sewer profile in upstream of Constable Street site

The trap chamber takes up the entire width of the sewer, with good access afforded via 5 manholes over a length of approximately 15 m. Chamber dimensions have been measured as 6.13 m long x 1.625 m wide x 1.233 m deep. This gives an overall capacity of 12.28 m^3, slightly less than previous sewer records show (13.4 m^3). A flow diversion arrangement is provided, using half gates at each end to divert the flow around the trap during maintenance.

This trap represents the largest of the traps in the Dundee area and is located near the foot of the Dundee system. As a result of this, the sediments arriving at this trap are a mixture of many particle sizes and types. This poses a good standard test for the use of invert traps as this could be considered "normal" or "average" conditions for a large drainage network.

3.4.2 Baldovan Road Silt Trap

The Baldovan Road site is located on a trunk sewer draining the North and East sides of the city in the Dighty catchment area. The trap is situated near the head of the system serving only a limited number of properties and a small hospital.

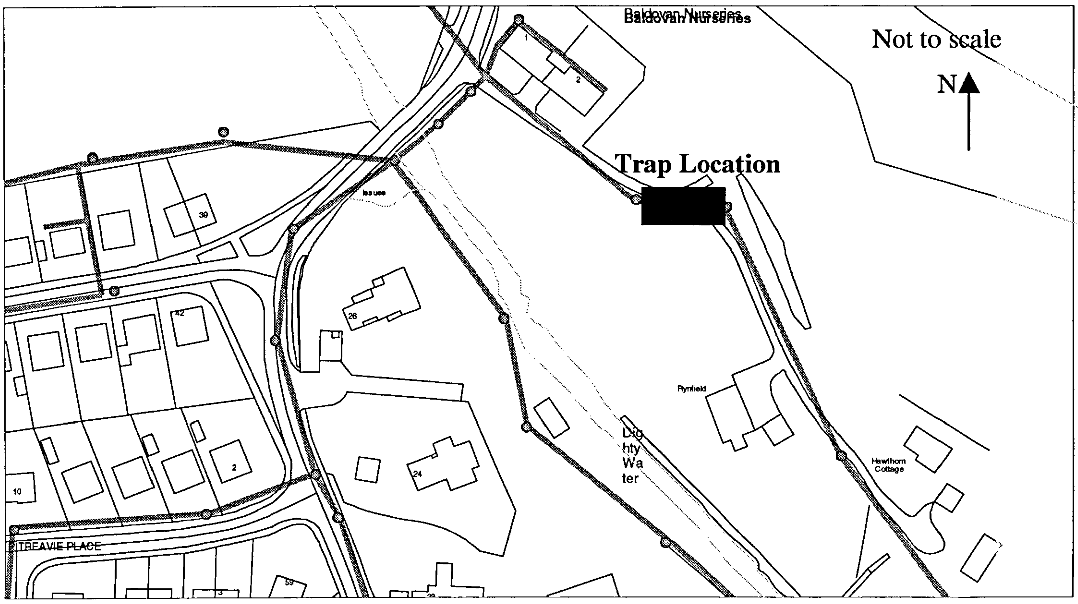


Figure 3.31 – Baldovan Road trap location

The incoming and exiting pipes are both circular, of diameter 900 mm. However, large differences can be seen to exist in the gradients of the pipes. The incoming pipe is steep with a gradient of 1:47, and the exiting pipe relatively slack at 1:320. It has been previously reasoned that this change in gradient represents the ideal location for a trap, as the change in hydraulics will aid the settling process (Ashley et al., 1995). The pipe levels around the trap are shown in Figure 3.33 (trap located at chainage=0m).

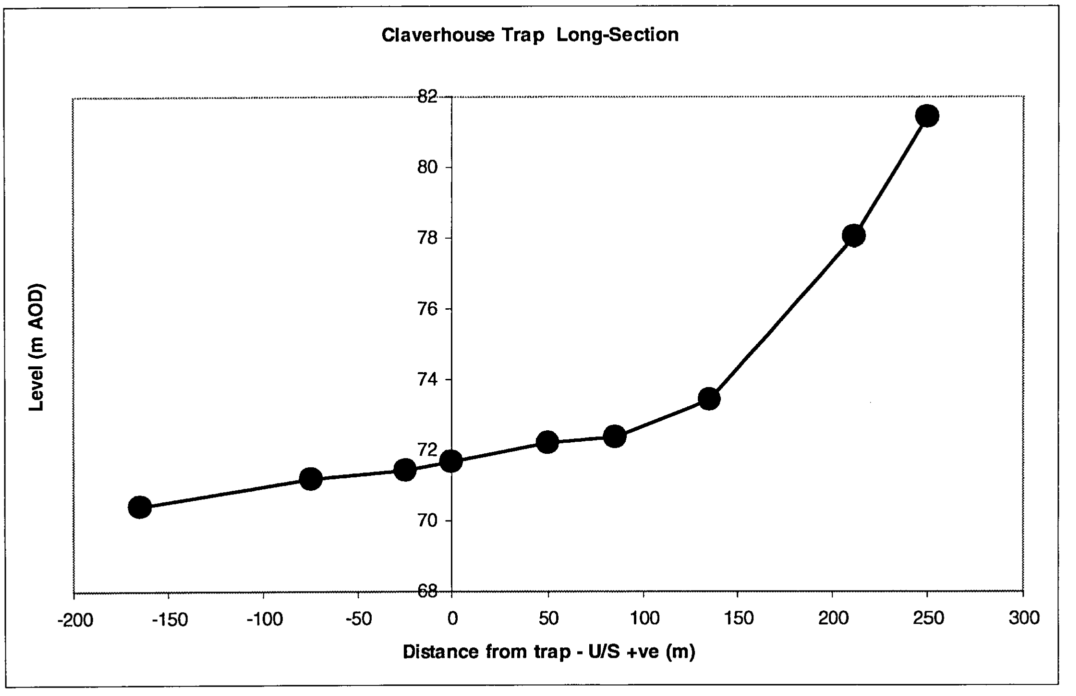


Figure 3.33 - Pipe long-section in vicinity of trap (trap at chainage 0 m)

The chamber is 3.2 m long x 0.85 m wide x 1.2 m deep, giving a capacity of approximately 2.88 m^3. The trap was constructed in 1995 and took account of sediment movement behaviour using a modified settling theory in its design. The modern design allows easy access around the chamber using a flow by-pass with walkway. The chamber's manholes are situated on a quiet access road with little disruption to local traffic during site visits. This site is characterised by low pulsing flows, and also the type of sediments that it receives. The bulk of the material arriving at this trap is granular and mineral in nature as a result of the low number of foul connections at this point. However, a significant catchment area is connected to the sewer at this point resulting in potentially high storm flows. The observation of this trap can therefore be used to assess the applicability of sediment traps to surface water drainage systems and the performance of a trap designed using settling theory.

3.4.3 Forfar Trunk Sewer Silt Traps

Two further modern traps are located at the foot of the Forfar drainage catchment. Attention was drawn to these sites by North of Scotland Water Authority, who had constructed these new traps but were unsure how to maximise their potential. The traps were constructed in response to the many sediment related problems experienced in the lower reaches of the Forfar sewerage system. The traps lie in close proximity to one another, on each of two parallel lengths of pipe carrying flow from the town centre to the treatment works.

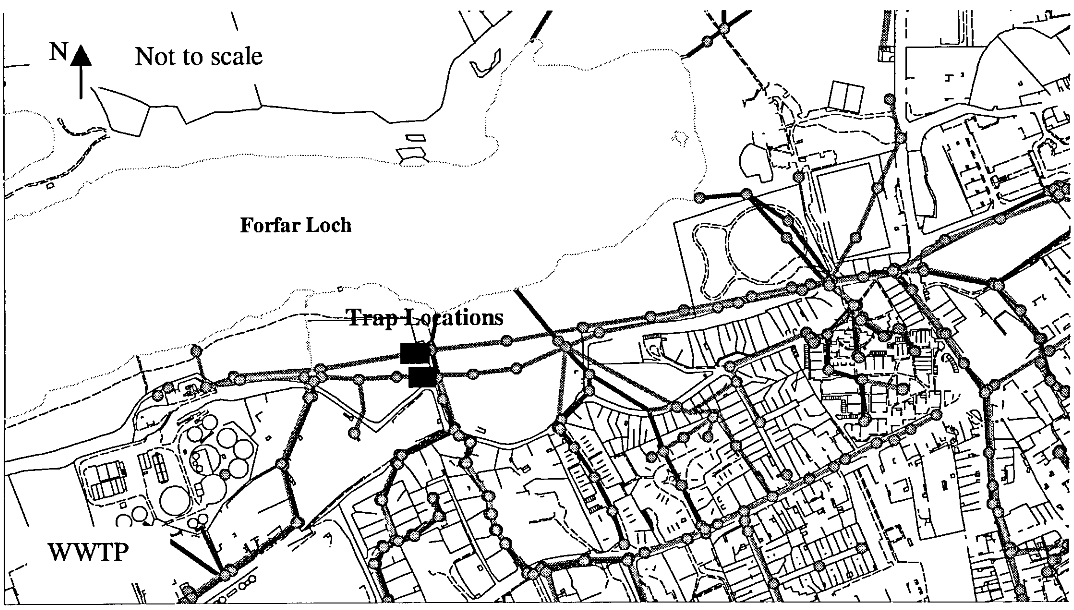


Figure 3.35 Forfar Trap locations

The hydraulics at the traps are quite complex with flow spilling from the main leg (900 mm diameter), to the auxiliary leg (600 mm diameter) in times of high flow. These complex hydraulics result in difficulties when representing the performance of the system using numerical modelling. These modelling problems are exacerbated by the slack pipe gradients at these sites and also the effects of pumps within the system, making general characterisation of the flows and sediments difficult. The observation of these traps offered the possibility of a full test of predictive and improved sediment methods to a real problem area. Figure 3.37 shows the incoming pipe levels to the trap on the 900mm sewer (trap at chainage = 0m).

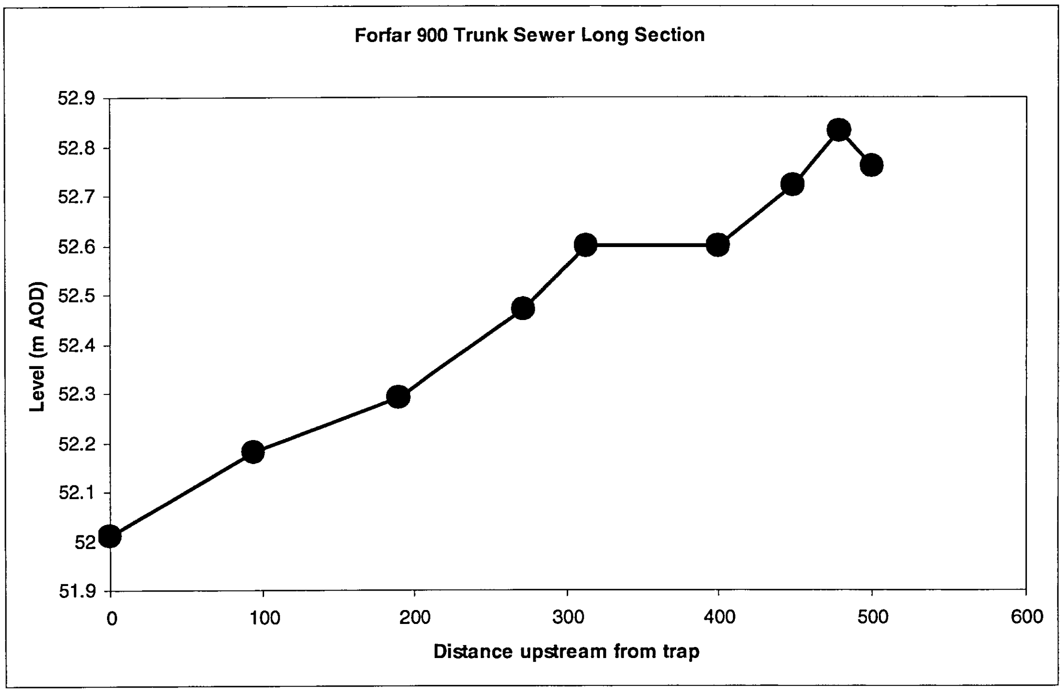


Figure 3.37 - Pipe long-section upstream from Forfar 900 trap

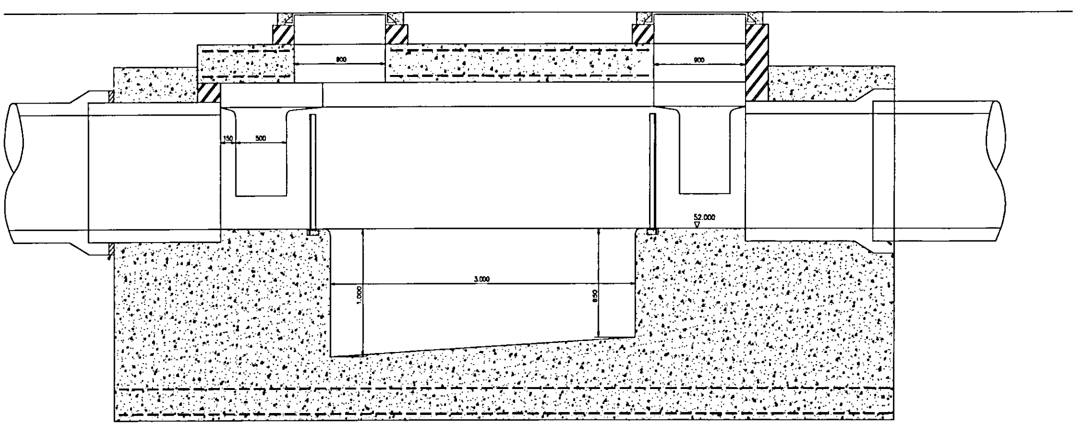


Figure 3.39 - Forfar trap long-section

Figure 3.39 shows a long section of the trap situated on the 900 mm pipe trunk sewer at Forfar. The inlet pipe to the chamber is on the right hand side, with a flow diversion channel shown at a higher invert level than the main pipes. Under normal operating conditions this channel is sealed off from the main flow channel using plastic, rubber edged, sliding gates. The trap base varies in level from 850 mm below invert level at the upstream end, to 1000 mm below invert level at the downstream end. This slope allows the preferential collection of trapped material and liquid directly below the access manhole for straightforward cleaning.

3.5 Data Collection Procedures

The following sections detail the procedures followed for the collection of the various data items.

3.5.1 Rainfall Data

As all the observed sites were subjected to storm inputs, rain data were recorded in the vicinity of all sites. In all cases, rainfall data were monitored using Casella tipping bucket style rain gauges.

The instrument employs a lightweight tipping bucket arrangement, fed from a metal funnel. The bucket mechanism is characterised by two tipping buckets each equating to 0.2 mm depth of rainfall falling in the funnel. The buckets are supported on a pivoting base, designed to reduce friction effects. At each tip of the bucket, a magnet housed within the bucket moulding closes a reed switch mounted in the support assembly. The pulses emitted thus represent a total of 0.2 mm of rainfall. These pulses are then time-stamped and recorded in an incorporated Technolog data-logging module.

The manufacturer's specification of the equipment is given as:

Tipping Bucket: 0.2 mm ±1%

Data Logging Module: Clock accuracy – 100 s/month maximum error

Although a regularly maintained tipping bucket rain gauge should provide reliable results, it should be noted that as the data are recorded at two minute intervals only average intensities over this period can be recorded. It is also unknown at the commencement of a rainfall event what volume of rainwater has been retained in the bucket from the previous rainfall event. This may result in an early first "tip", indicating a more intense start to a rainfall event.

3.5.2 Sewer Flow Data

Prior to any analysis of sediment transport rates, a full understanding of the flow characteristics at all selected sites is required.

Two types of flow data were collected as part of this study:

- Time varying, average flow depths and velocities recorded in pipe at two-minute intervals;
- Detailed velocity profiling carried out throughout the flow depth either in pipe or within the sediment traps.

3.5.2.1 Flow Survey Equipment

The principal piece of equipment used to provide time varying flow data was a DETEC IS Survey Logger (Model number 3510). The unit comprises three main modules:

1. The transducer head which is installed on the pipe invert facing oncoming flows;
2. The data logger which is housed in an intrinsically safe casing and suspended in the adjacent manhole;
3. The battery unit used to power the logger, which is also housed within the intrinsically safe case.

The transducer head contains velocity and depth sensors which are used to calculate the total flow rate. The average velocity of the flow is calculated using the Doppler effect. A transmitter emits a fixed frequency, ultrasonic signal, which is reflected by particles and air bubbles in the flow. This reflected signal is returned at a different frequency depending upon the velocity of the particles intercepted. This shift in frequency is then converted by the unit into an average flow velocity. At the same time that the velocity is recorded, a pressure transducer in the head converts the recorded pressure into an equivalent depth of water. The measured depth and velocity are then recorded simultaneously and time-stamped at a given interval.

The manufacturer's specification of the equipment is given as:

Velocity Transducer

Range: 0.1 to 4 m/s	Resolution: 1 mm/s	Accuracy: ±2.5%

Depth Transducer

Range: 0.05 to 2 m	Resolution 1mm	Accuracy: ±0.3%

It should be noted that these figures are provided for "ideal" flow conditions. In reality, depth and velocity data cannot be recorded with any reliability in flows less than 50 mm deep. Additionally, the accuracy of flow measurement has also been observed to reduce significantly in flow velocities greater than 2 m/s. The accuracy of flow recording is therefore most problematic in shallow, fast ambient conditions.

3.5.3 Sediment Depth

As a consequence of the varying accessibility to the required field investigation sites, various forms of sediment level measurement were used.

3.5.3.1 Direct Measurement

In areas where man entry was permitted along the full length of the sediment samples (either in-pipe or sediment trap), direct, physical measurement was used. For the Baldovan Road trap, a gantry exists along the full length of the trap. This gantry allowed access to the trap along its full width and length. Water levels were recorded at the inlet and outlet channels and all sediment depths measured relative to the water level over a grid at 0.25 m (transverse) and 0.5 m (longitudinal) spacings.

For in-pipe sediment deposits where man-entry was afforded, the sediment depth was recorded using a base plate and depth gauge arrangement. Figure 3.41 shows the mode of operation for the base plate gauge. The base plate is first brought into contact with the water/sediment interface. Care should be taken at this stage not to compress the sediment sample. The gauge needle is then driven through the sample and the graduated scale (previously zeroed for the bottom of the base plate) read to give the sediment depth.

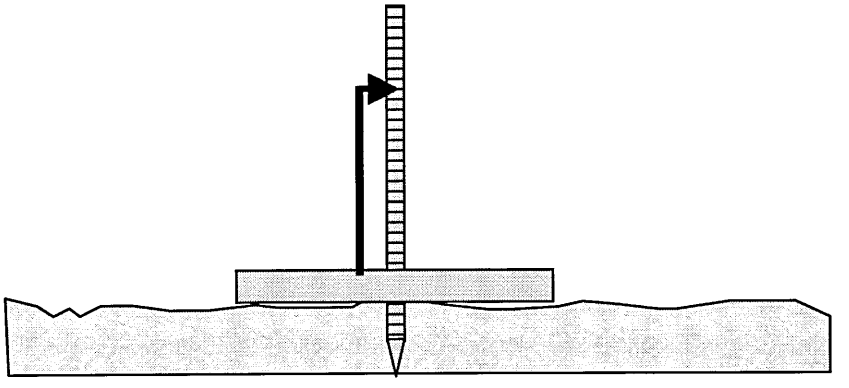


Figure 3.41 - Base plate sediment depth gauge

For both methods of direct measurement, there is only limited scope for significant error. The simplicity of the methods and the fact that a series of independent readings is generally taken means that gross errors are easily identified at the time of measurement and any measurements can be readily repeated.

3.5.3.2 Sonar Measurement

In areas where man entry was not available or practical, sonar methods were generally used.

3.5.3.2.1 PypScan

At locations where direct measurement of sediment deposits was either impractical or undesirable, the PypScan system was used to give a snapshot measurement for a given site visit.

The system was principally used to determine the location of the sediment/water interface for the more inaccessible invert traps but was also used on occasions to investigate pipe deposits. The unit comprises an underwater scanner (mounted on a float), connecting cable drum, sonar processor, industrial computer and display monitor.

3.5.3.2.1.1 PypScan Laboratory Testing

Prior to use in the field, the system was used in a controlled laboratory condition in order to provide information on the accuracy of the method of measurement and the level of resolution attainable.

The sonar unit was suspended just below the water surface in a trapezoidal plastic tank, using a wooden frame (Figure 3.43).

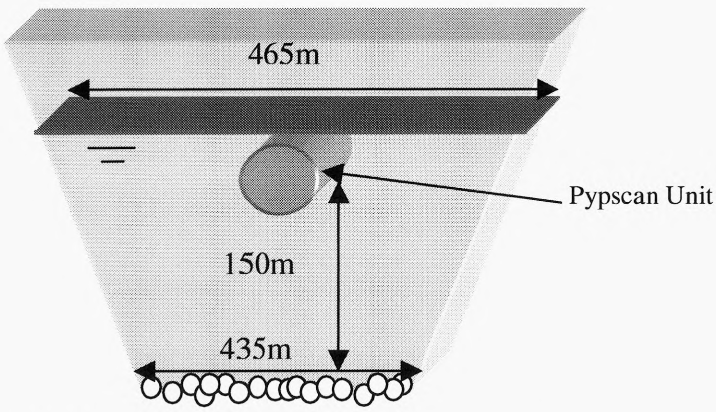


Figure 3.43 – PypScan laboratory testing

The first tests carried out were used to determine the reliability of the distance measurements quoted by the unit. A layer of coarse gravel was placed in the bottom of the tank and the readings from the unit compared to actual physical measurements. Within a tested range of 50 to 250 mm, all sonar readings were found to be accurate to within 4 mm in all cases. The surface profile of the gravel was also easily detected and individual stones could be easily identified.

On the basis of these readings it was determined that the standard method of calibrating readings (adjustment of velocity of sound used for calculations) was not required.

The unit was then tested for the range of materials that it was able to detect. A layer of low density sponge material (S.G. = 1.37) was soaked and positioned on the bottom of the tank. The section was scanned, and with limited adjustment of the display parameters the following measurements could be made.

Actual Dimension (mm)	Scanned Dimension (mm)
Breadth = 150 mm	Breadth = 152mm
Depth = 25 mm	Depth = 24 mm

Table 3.11 – Performance of low density scans

The above results show no loss in accuracy when using the low density material. The depth of the sponge object was ascertainable as a result of the permeability of the material. The sonar system was able to penetrate the sponge highlighting its shape and was even able to detect some of the more protruding stones below the sponge layer. In order to determine the minimum thickness of low density material that could be detected, a single layer of laboratory tissue roll was suspended in the scanning area. Although the sheet of roll was less the 0.5 mm thick and of low density (S.G. = 1.25), the scanner clearly determined the dimensions, and shape of the sheet and could, to a limited extent, penetrate through the sheet.

Although the use of sponge provided a low density material to be scanned, it differed from organic sediment predominantly in that a well defined material boundary does not exist with very low density sediment. In order to provide a more realistic representation of a graded interface, sawdust was allowed to settle within the tank over a 24 hour period. This provided a very fluid low density material to be scanned. In all cases, the accuracy of measurement was similar to that experienced for the gravel and sponge tests.

As a result of the potential for using the sonar unit in high solids loading (e.g. Forfar invert trap sites), a test was carried out to ascertain the ability of the equipment to scan through turbid waters. The unit was set up in the original configuration in the tank, with a layer of gravel on the bottom. Scans were then taken continuously, whilst sawdust was added to the tank. The sawdust-water mixture was then continuously agitated to keep the sawdust in suspension. The sawdust was added until the point that the gravel's surface was no longer visible to the pipe profiler. At this point, samples were extracted using a suction method and tested for suspended solids. This test was carried out 6 times, producing the results given in Table 3.12.

Test No.	Critical Conc.
1	373 mg/l
2	420 mg/l
3	397 mg/l
4	428 mg/l
5	409 mg/l
6	422 mg/l

Table 3.12 - Critical solids concentration tests

These results give an average critical suspended solids concentration of 408 mg/l (s.d. 18.65 mg/l). This represents approximately twice the suspended solids concentrations experienced at the field test sites during dry weather flow. Consequently, the turbidity of the flow is not perceived to represent problems for standard scans of sediment traps.

The pipe profiler ably coped with the variety of conditions it was presented with, and also demonstrated an ease of use and scan speed which makes scanning in the difficult conditions associated with sewer measurement relatively simple.

At each trap site, the sonar unit was lowered into the upstream manhole of the trap. Personnel were located at each end of the trap in order to manoeuvre the floating unit within the trap. Scans were then taken to give transverse profiles of the sediment trap at longitudinal spacings of either 0.5 or 1 m (depending upon trap size). Each scanned image was saved using a file name denoting the date and position of the scan. In this way, a three dimensional record of the filling of the trap can be determined (Figure 3.45).

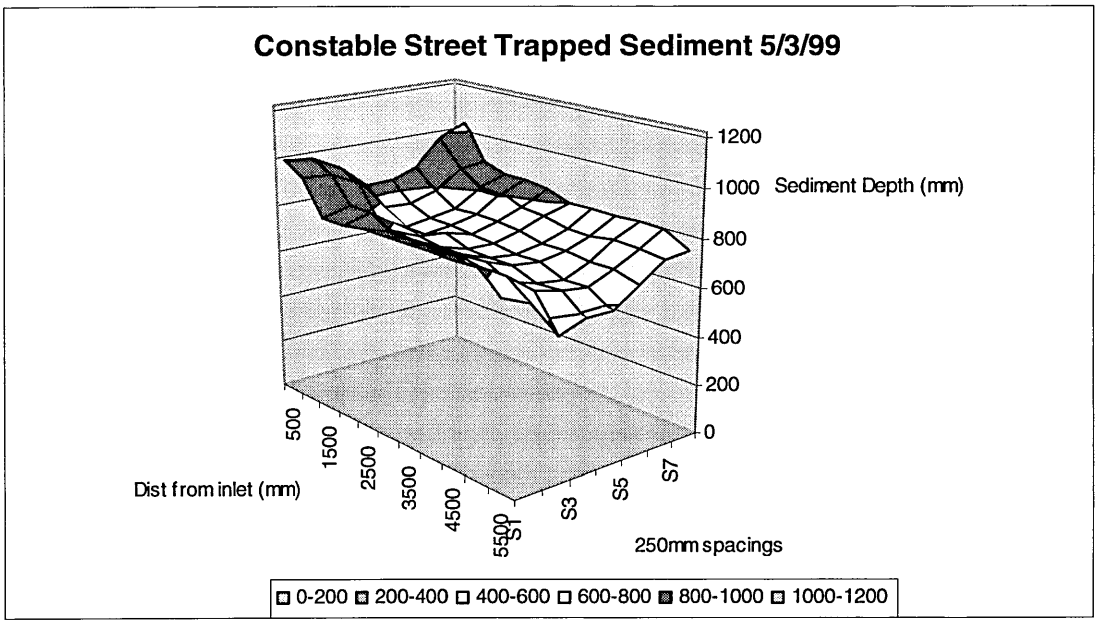


Figure 3.45 - Example of data received from sonar scans

3.5.3.2.2 Fixed Sonar Units

In addition to the determination of sediment depths at weekly intervals, equipment was developed and tested to enable automated, frequent measurement of sediment levels in traps or pipes. The principal driver of the development of these units was to enable sediment depths to be recorded during storm conditions. Consequently the units must be able to record data at a sufficient time resolution to allow this. The design of the units was based upon that used successfully in previous studies (Wotherspoon, 1994), with some minor modifications made in an attempt to enhance the stability of the readings. The principal modification made to the unit involved the replacement of the solid state mounting of the signal crystal with an automatic levelling system mounted in oil. This work was carried out as a result of previous difficulties in aligning the unit vertically.

The units comprise:

- Sonar head – containing sending and receiving crystal for sonar detection;
- Data transmission cable;
- Data processing unit – containing logic circuits and electronic circuits;
- Data logging unit – stores logged data at 2-minute intervals.

The units were developed and rigorously tested in the laboratory prior to field installation. For each unit, tests of accuracy of measurement, range, material sensitivity and turbidity were carried out using a similar set of tests used for the scanning sonar. Following these tests, an optimum calibration setting was determined for each unit by adjusting the various control circuits in the data processing unit. Following this, a calibration plot of recorded voltage versus measured depth was created to allow sediment depths to be determined from the signal returned by the unit. Figure 3.47 shows a sample of the type of plot created during the calibration process. This figure clearly shows an alteration on the behaviour of the unit at a depth of 250 mm. Consequently should depths of less than 250 mm be required to be measured, re-calibration of the measurable range should be carried out.

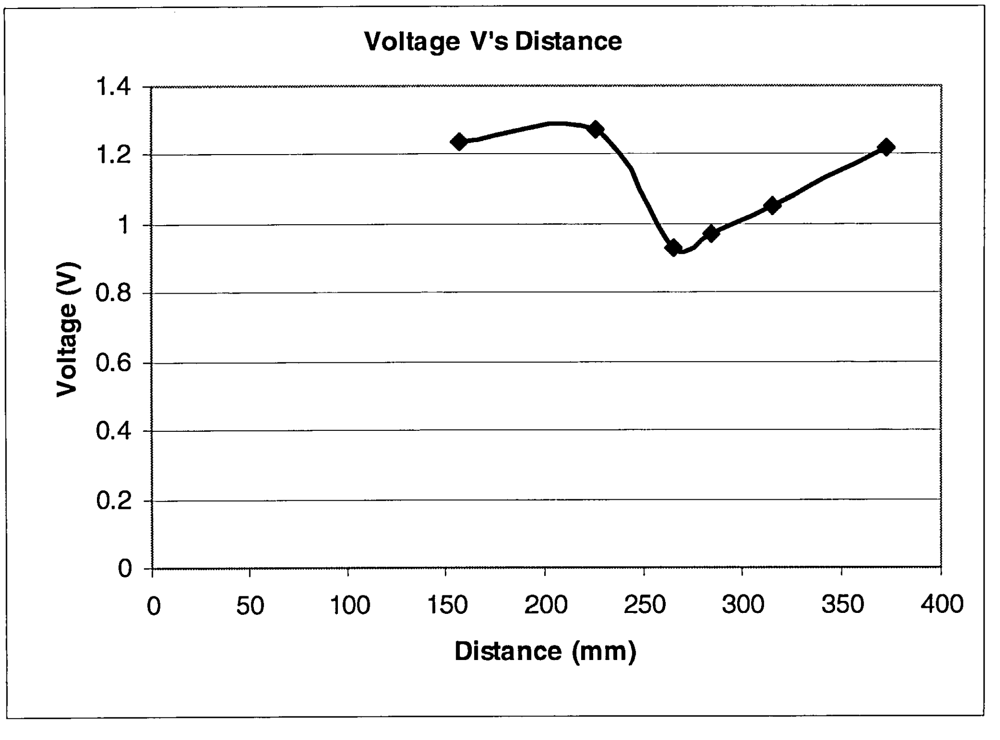


Figure 3.47 - Sample sonar head calibration plot

In order to test the operational parameters of these units they were tested in the laboratory for performance under the following varied conditions.

- Angle of incidence to flat plate;
- Type of material detected;
- Effects of turbidity.

It is essential in the setting up of the equipment that the sonar emitting crystal is aligned at right angles to the direction of detection. This alignment sensitivity was highlighted as a problem and was addressed by altering the head design to incorporate a self-levelling crystal mounted in oil as opposed to the traditional fixed mounting in epoxy resin. The fixed sonar units were found to operate satisfactorily for the intended processes but were still found to be sensitive to alignment problems. Although the new arrangement improved the initial setting up of the unit markedly, the angle of incidence tests showed an improvement of between 10 and 30 %. The actual critical angle of incidence was found to vary directly with the distance to the angled plate. This is believed to be associated with the spread of the sonar signal as it travels through water, gradually increasing the size of the sonic "beam". Hence at larger depths, the wider reflected beam is more easily detected by the unit.

The fixed units were found to detect base material down to a density of approximately 1600 kg/m^3. Although this does not match the level of performance of the scanning sonar, the site selected for the installation of these units contains a significant proportion of material above this density.

The critical suspended solids concentration was more difficult to define for the fixed sonar units as the value was found to vary widely between units and also for each test (Table 3.13).

On the basis of all tests, units 1 and 2 were selected for use in the field, with units 3 and 4 retained as back up.

Test Number	Critical Conc. For Head 1 (mg/l)	Critical Conc. For Head 2 (mg/l	Critical Conc. For Head 3 (mg/l	Critical Conc. For Head 4 (mg/l
1	181	254	265	152
2	254	286	156	109
3	321	224	185	75
4	122	319	192	156

Table 3.13 - Critcal sediment concentrations for fixed sonar measurement

3.5.4 Sediment Transport Rate

The rate of sediment transport was determined in two ways in order to measure the two principal modes of sediment transport (suspended and bed movement)

3.5.4.1 Suspended Solids Sampling

Traditionally, suspended solids concentrations in sewer flows have been determined through the use of suction samplers. In this process, a sampling tube is weighted and inserted into the flow. Samples are then extracted from the flow at predetermined intervals and deposited into various sample bottles via a vacuum chamber. However, there are a number of uncertainties associated with this process:

- The sampling process has been observed to be selective, as only particles with diameters less than the tube diameter (approximately 10-mm) can be sampled. Although in general samples of this type are not present as suspended material, this limits the sampling to exclude gross solids and larger material that may be present near the pipe invert.
- The sediment concentration is known to vary significantly with depth. There is therefore no way of ensuring that samples taken are representative of conditions throughout the flow column.
- The sampling point within the flow column is unknown and will vary with hydraulic conditions.
- As the sample tube is generally located near the pipe invert, any bed material present can be sampled and mistaken for material in transport.

In order to address these limitations, fixed point, multi-depth sampling was undertaken. A rigid PVC cylinder was used to house the multi-depth tubes in the centre of the flow column. Holes were drilled at various heights to allow the sample tubes to protrude from the main column at varying depths. The distance over which the sample tubes protruded was varied so as to minimise disturbance between adjacent samples in the event of simultaneous sampling. Figure 3.49 shows a diagrammatic representation of the system used.

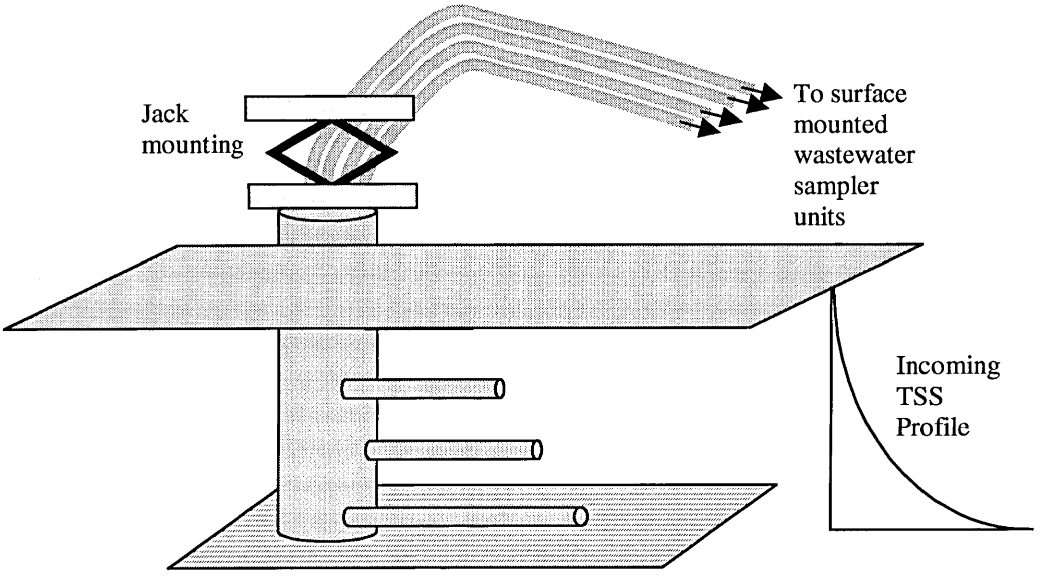


Figure 3.49 - Multi-depth suspended solids sampling equipment

The base of the unit was mounted on a length of aluminium plate in order to reduce the chance of abstracting bed material. The unit was topped with a jack arrangement, which allowed it to be securely fixed in place between the pipe invert and soffit.

3.5.4.2 Bed-load / Near Bed Solids Sampling

The presence of the trap at each site allowed samples of bed material to be taken using small scale, removable bed traps. These traps were sized to trap all material moving near the bed through the calculation of particle jump lengths and the visual observation of the dry weather material at each of the sites.

In cases where material was already present in the main trap, the sediment was excavated to allow the removable traps to be temporarily placed at the inlet pipe, with the top of the traps at the same level as the inlet pipe invert. The traps were then left for typically 4 hours at a time in order to estimate the rate of sediment movement (and characteristics) at various times of the day.

3.5.5 Sediment Deposit Sampling

In addition to collecting data on the location and quantity of sediment deposits (both in-pipe and trapped), it is necessary to obtain samples of these deposits in order to gain more information regarding their characteristics. The method used to obtain these samples typically depended on the source of the sample, its depth, the number of samples required and the particular purpose of the sampling. Sediment deposits have been shown to be highly variable, with potentially complex and stratified structures. If these structures are to be retained, a modified sampling procedure must be employed.

3.5.5.1 Direct Sampling (disturbed)

In areas where only surface samples or a large number of bed samples were required direct sampling was undertaken. In the case of bed sampling, a wide necked sample container is inserted into the bed until the neck is in contact with the pipe invert. The container is then thrust in an upstream direction so as to "scoop" as large a sample as possible in a single movement. Where surface samples of trapped sediment were required, the sample containers were mounted on stiff sample rods and the same process repeated using the rod and container combined.

3.5.5.2 Cryogenic Coring

The direct sampling method has two principal drawbacks:

1. Only surface samples may be taken;
2. The sample is entirely disturbed and the relative positions of sediments within the sample are completely lost.

These drawbacks become significant when extracting samples of trapped sediments as deep samples are required along with the position of sediments to give a history of trap performance and details of any stratification that may take place as a result of changing trap performance during trap filling.

Consequently, coring techniques were investigated which may be suitable to be applied to sewer sediment data collection. The principal techniques investigated were:

- Cohesive binder injection;
- Negative pressure sampling;
- Cryogenic sampling.

The cohesive binder injection method involves isolating the desired sample area using a corer, then injecting the sample with a binding solution. This method was discounted at an early stage as the process would prevent any meaningful quality laboratory tests being carried out, as even once the glue is later dissolved by a secondary agent, biochemical tests are often affected.

Negative pressure sampling involves the construction of a plunger arrangement within a coring tube. The tube is inserted until the plunger is immediately above the water's surface. The plunger is then raised to reduce the pressure above the sample and assist in the retention of the sample within the tube. Tests of this method demonstrated its usefulness only for stiff cohesive samples, as any pressure reduction is quickly lost through more porous samples. In addition to this, to be effective, a relatively small core diameter should be used and the structure of the core is not accurately maintained following extraction from the corer.

A cryogenic corer was designed using the experiences of fieldwork testing in Hannover, Germany (Ristenpart & Uhl, 1993). The unit comprises an external PVC corer with an internal freezing core of stainless steel tubing. The freezing medium (a mixture of dry ice and methanol) is inserted into the central core in order to freeze the pore water of the surrounding sample. The frozen core is then extracted and can be taken back to the laboratory intact for examination, dissection and sample analysis.

The procedure was initially tested in the laboratory to assess the suitability of the unit to field activities. A manufactured sediment bed was carefully constructed within a laboratory tank. A stratified structure was simulated in order to assess the effects of freezing the sample within the corer. Known depths of cohesive mud, grits and sawdust were layered to produce an overall sample depth of 250 mm. This process was carried out slowly under a depth of water. Three cryogenic cores were then taken from the artificial bed and compared to the recorded sample depths as constructed.

Core Characteristic	Constructed Dimension (mm)	Cryogen Core 1 Dimension (mm)	Cryogen Core 2 Dimension (mm)	Cryogen Core 3 Dimension (mm)
Overall Depth	250	255	256	252
Mud layer 2 thickness	30	32	30	33
Grit layer 1 thickness	40	40	38	42
Sawdust layer 2 thickness	25	28	26	24

Table 3.14 - Cryogen core testing

The testing showed the structure of the core to be generally maintained with only a minor expansion noted, with all layers easily visually discernible. On the basis of this testing, the procedure was deemed acceptable for application to drainage sediments.

3.5.6 Sediment Quality

A range of tests was carried out in order to characterise the sediment samples extracted as part of this study. These tests served two principal purposes:

- To further understand the physical processes and behaviour of sediments at each of the sites;

- To use (where appropriate) these characteristics as default values for modelling exercises.

The tests carried out are described further in the following sections.

3.5.6.1 Particle Size

A particle size analysis was carried out on all samples using one of the two methods detailed below:

1. Dry sieving of the mineral fraction in accordance with BS1377 (BSI, 1975);
2. In order to ascertain the particle size distribution for the finest and/or organic fractions present in sediment samples the "Malvern Mastersizer" was used. This proprietary piece of equipment uses the data collected on the characteristics of a refracted laser beam as it passes through a suspended sample to achieve a correlation with suspended sample concentrations and sizes.

3.5.6.2 Settling Velocity

The settling velocities of samples were estimated using the UFT method (Michelbach & Wohrle, 1992). Further details of this method are provided in Appendix C.

3.5.6.3 Sediment Concentration

The sediment concentration of a given sample was typically measured using the sample weights before and after drying at 105 °C. In the case of sediment samples, these weights were for bulk samples, and in the case of suspended material, the samples were filtered before drying in accordance with standard laboratory procedures. In addition to this method, the Mastersizer was also used for fine low-density suspended material.

3.5.6.4 Organic Content

The organic fraction of the samples was determined through the measurement of dry sample weights before and after the removal of the organic material (through furnacing at 550 °C). The organic content is then expressed as a fraction of the original dry mass of the sample.

3.5.6.5 Sediment Density

Sediment density is an important characteristic, as this plays a significant role in most transport relationships. For each sample taken, both the bulk and dry densities were determined. The bulk density was determined through the direct measurement of the mass and volume of each sample, with the dry density determined using the original volume but the weight of dry solids only.

3.5.6.6 Sediment Polluting Potential

In addition to the determination of the physical characteristics of the samples extracted, the polluting potential of the material was also determined through the measurement of BOD_5, COD and ammonia concentrations.

3.6 Data Collected

The methods described in the previous sections were used to collect as broad a range of information as possible from each selected site. In addition to the details provided below, hydraulic data (sewer flows and rainfall) were collected continuously throughout the data collection period at all sites.

3.6.1 Sediment Trap Fill Volumes

Sediment trap fill volume data were used for the calibration of the sediment trap model and for general observations of sediment movement over long durations. As a consequence of the variability of site access, the volumes of sediment retained in the traps were measured using a method of direct measurement at the Baldovan Road site and using the scanning sonar at the Forfar and Constable Street sites.

The periods over which data could be collected at each of the sites were essentially dictated by the planned maintenance schedules of Scottish Water, as the cleaning of the traps was required before measurements could be taken. As a result of this, and the time taken for the traps to fill, only a restricted number of data-sets could be collected. Two fill patterns were recorded at the Constable Street and Baldovan Road sites, and three patterns collected at the Forfar site (as the trap filled quickly). The additional fill pattern collected at the Forfar site was carried out to assess the performance of a modified trap configuration. Further details of this configuration are given in Section 3.6.1.3.

3.6.1.1 Constable Street Fill Patterns

The fill patterns obtained through the measurement of the sediment surface in the Constable Street sediment trap are shown in Figure 3.51 and Figure 3.53. In both cases, the traditional assymptotic curve profile is evident, with an initial average fill rate of 0.075 m^3/day. In both cases, this fill rate was observed to decay towards zero from a trapped volume of approximately 7.5 m^3. This point cannot accurately be picked out from fill pattern 2 as a complete data set was not available.

The ultimate retained volume in the trap was found to vary between 9.8 and 10.4 m^3, depending upon the hydraulic conditions prevelant immediately prior to measurement.

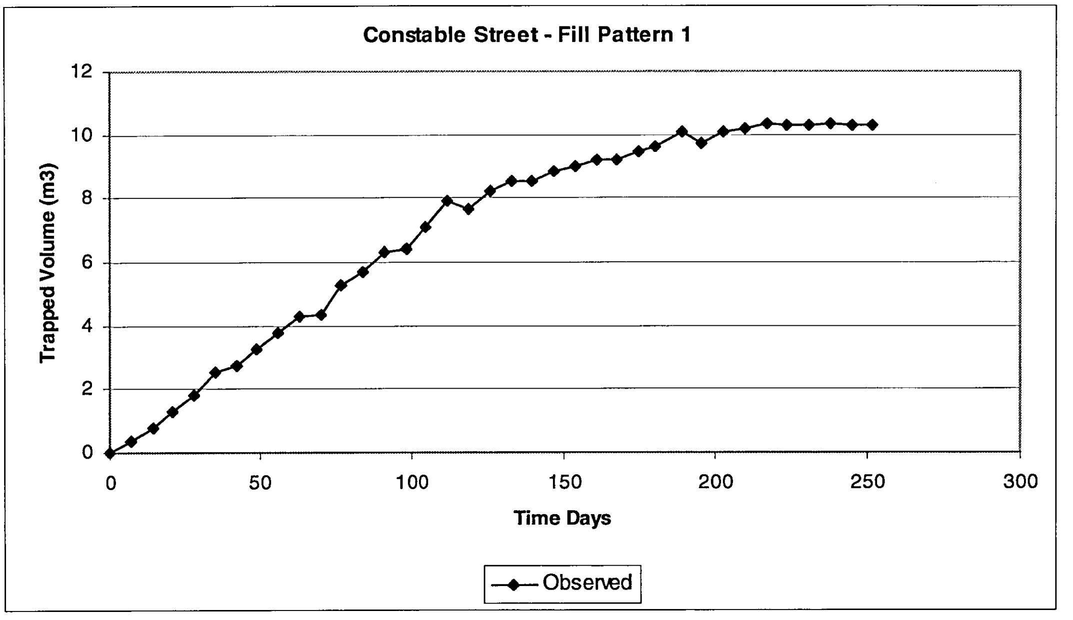


Figure 3.51 - Constable Street sediment trap fill pattern 1

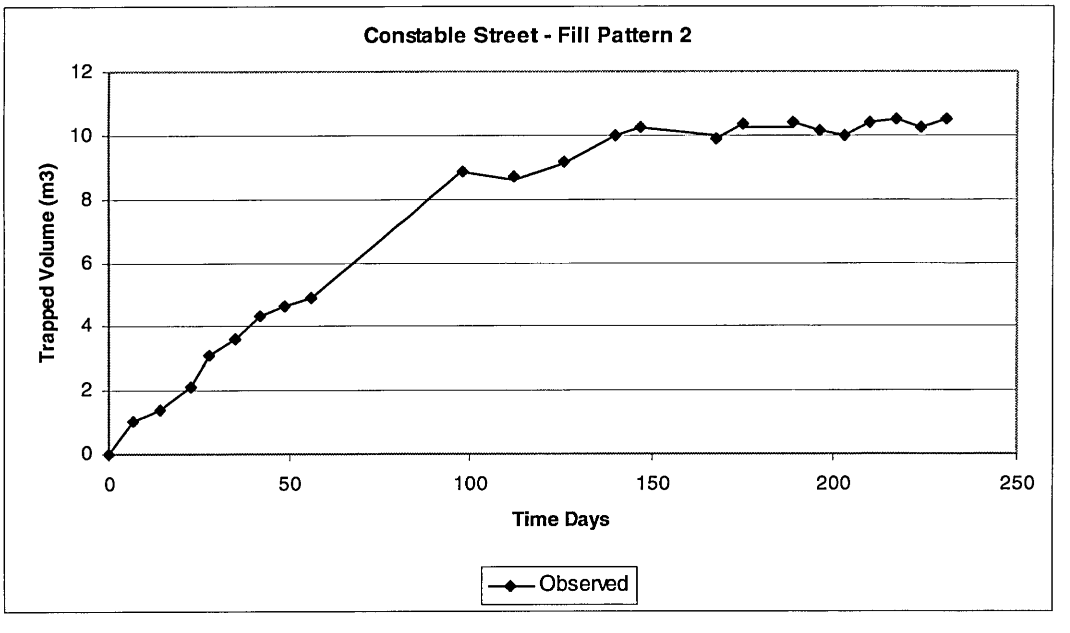


Figure 3.53 - Constable Street sediment trap fill pattern 2

3.6.1.2 Baldovan Road – Claverhouse Fill Patterns

Two sets of trap filling data were collected at the Baldovan Road site. These patterns are shown in Figure 3.55 and Figure 3.57. As can be seen, both patterns are characterised by a high initial fill rate, followed by a gradual decline as the trap's capacity is reached. It should be noted that the patterns of this site are more significantly affected by rainfall conditions with occasional sudden jumps (or in some cases drops) found after heavy rain. As a result of the direct method of measurement employed at this site, a high degree of confidence in the data exists.

In both cases, the trap was observed to fill to approximately 75% of its volume. This is lower that the filling percentages found at the other two sites and is believed to be a result of the higher ambient velocities experienced at this site.

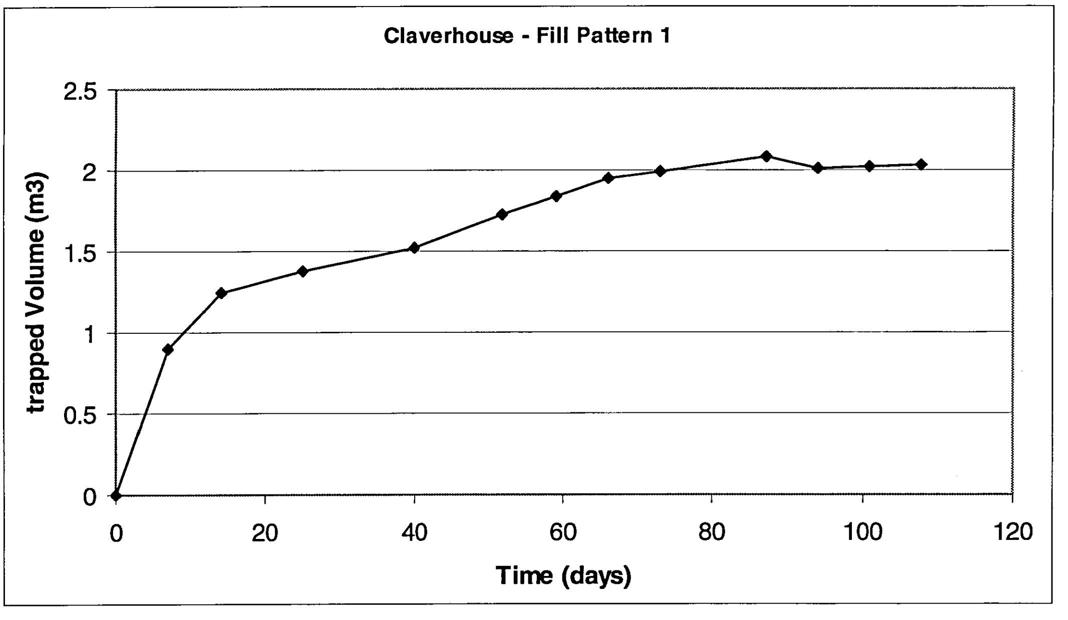


Figure 3.55 – Baldovan Rd - Claverhouse sediment trap fill pattern 1

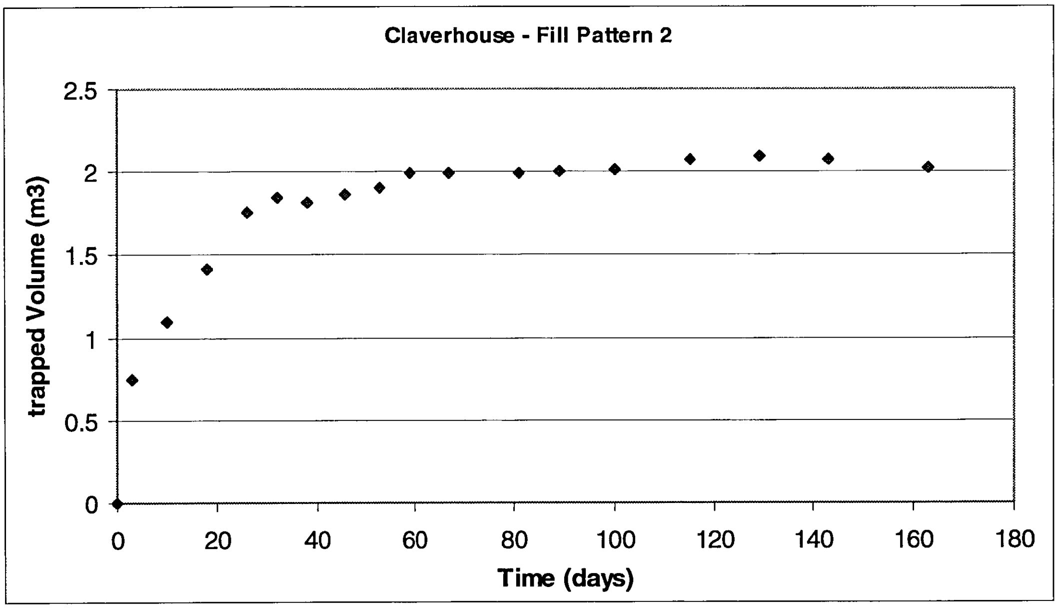


Figure 3.57 – Baldovan Rd - Claverhouse sediment trap fill pattern 2

3.6.1.3 Forfar 900 Fill Patterns

Three sets of filling data were collected at the Forfar site. The first two data sets were taken for the original hydraulic regime at the site found at the outset of the study. Under these conditions the trap was found to fill very rapidly. With the ultimate capacity of the trap reached within approximately 20 days. It was found in both of these fill patterns that 100% filling was always exceeded as the sediment bed depth upstream and downstream of the trap was quickly found to form over the trap. Each of these curves is characterised by a very rapid initial fill rate and a sudden plateau after between 7 to 14 days. These patterns are shown in Figure 3.59 and Figure 3.61.

A programme of hydraulic improvements was carried out at the site in an attempt to enhance the performance of the traps and reduce the level of sediment deposition within the trunk sewer as this was found to impact the operation of the Wastewater Treatment Plant and restrict network capacity. Further details of these improvements and their effects are given in Section 3.6.7. These hydraulic improvements resulted in improved trap performance. As can be seen in Figure 3.63, the initial fill rate is somewhat reduced and a more gradual decline in fill rate is therefore observed. The capacity of the trap is not fully reached until approximately 40 days. This is twice the period experienced prior to the hydraulic improvements being implemented.

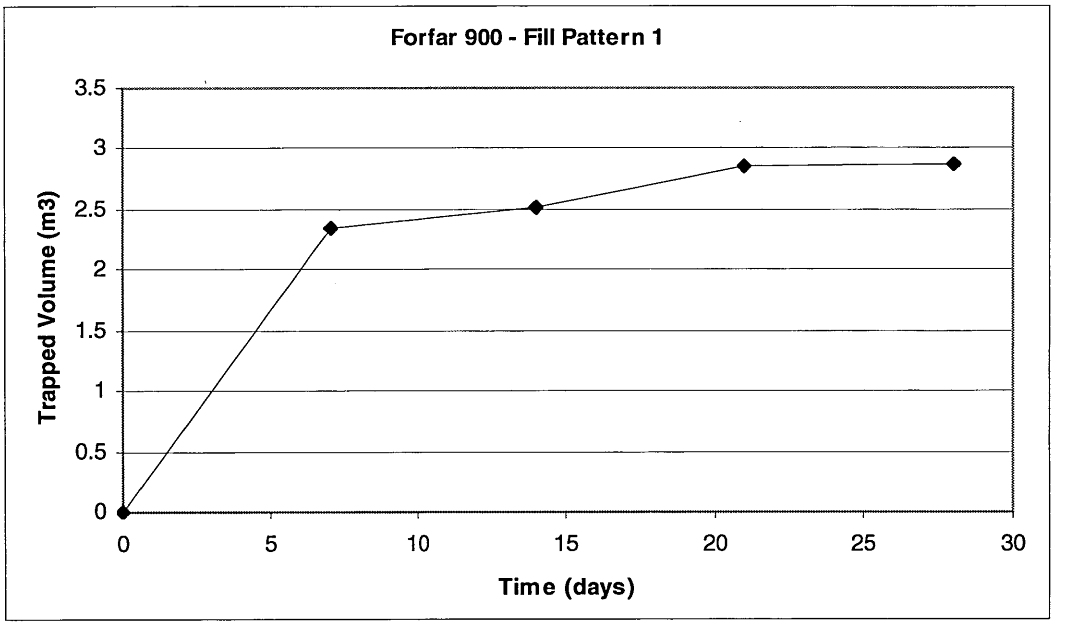


Figure 3.59 – Forfar 900 sediment trap fill pattern 1

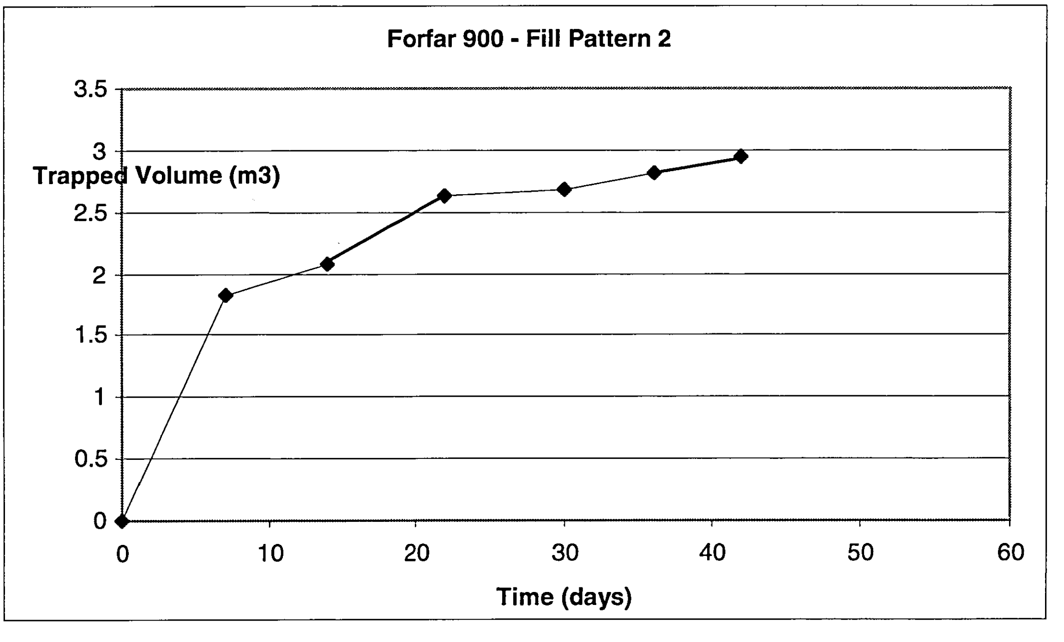


Figure 3.61 – Forfar 900 sediment trap fill pattern 2

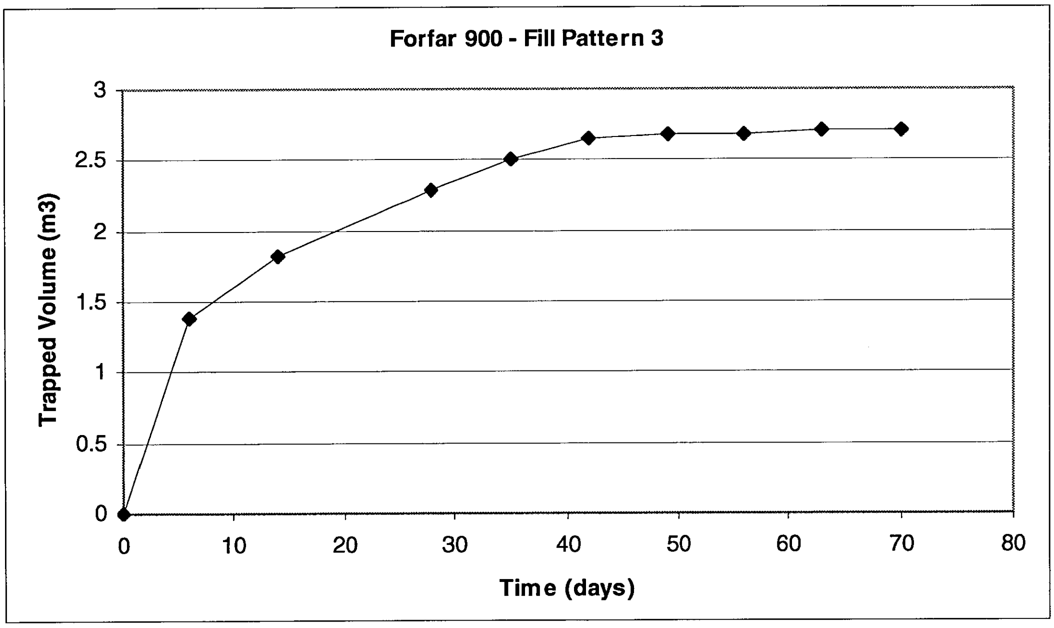


Figure 3.63 – Forfar 900 sediment trap fill pattern 3

3.6.2 Trapped Sediment Characteristics

In order to represent the filling rates of the traps using a physically based model it is necessary to determine the physical and bio-chemical characteristics of the material that is retained within the trap. This also allows an assessment of the performance of the trap to be carried out.

3.6.2.1 Disturbed Surface Samples

During trap filling, surface samples were taken periodically in order to establish the characteristics of the material being trapped and if those characteristics varied within the trap. In addition to this, samples were also taken of pipe deposits in the immediate trap vicinity. The location, frequency and number of samples taken were dependent on the accessibility of each site.

3.6.2.1.1 Constable Street Trapped Sediments

Although access could only be gained at the ends of the trap, the stability of the trapped material allowed samples to be taken up to approximately 5 m into the trap length.

During the early stages of trap filling, the material sampled at the trap inlet was observed to be a mixture of small gravels and finer organic material. The overall d_{50} of this mixture was found to be smaller than that of the upstream bed deposits. As the trapped deposits built up, the trap inlet deposits became more comparable with those of the upstream pipe with a gradual reduction of particle size and increase in organic content with increasing distance downstream. As the trap continued to fill, the surface samples became more consistent with the pipe deposits at all points in the trap. These results are summarised in Table 3.15. It should be noted that a rainfall event was recorded on the day prior to day 105.

Day	U/S Pipe d_{50} (mm)	0 m d_{50} (mm)	1.5 m d_{50} (mm)	4.5 m d_{50} (mm)
14	4	1.25	3.35	0.60
105	3.35	2.75	3.20	1.80
245	3.75	3.50	2.85	2.15

Table 3.15 - Particle sizes for Constable Street Fill Pattern 1

As can be seen, the samples reveal a significant variation in the nature of trapped sediment both with the location of the sample and the time at which it is taken. Consequently, previous studies, which have concluded that there was little difference between trapped sediments and pipe deposits (Fairweather 1995; Sutherland 1996) are erroneous as the samples in these studies were taken as the traps neared their full condition. The sampling carried out as part of this investigation reveals that at this stage of filling the surface samples are nearing the characteristics of the upstream pipe deposits as a consequence of similar hydraulic regimes. Further characteristics of the Constable Street samples are shown in Table 3.16 and Table 3.17.

Day	U/S Pipe % volatile	0 m % volatile	1.5 m % volatile	4.5 m % volatile
14	2	6	2.5	8
105	2	4	2.5	6
245	2	3.5	2.5	5.5

Table 3.16 – Volatile content for Constable Street Fill Pattern 1

Day	U/S Pipe ρ_b (kg/m^3)	0 m ρ_b (kg/m^3)	1.5 m ρ_b (kg/m^3)	4.5 m ρ_b (kg/m^3)
14	1911	1674	2008	1509
105	1958	1862	1951	1604
245	1922	1874	1895	1566

Table 3.17 – Bulk density for Constable Street Fill Pattern 1

3.6.2.1.2 Baldovan Road - Claverhouse Trapped Sediments

As access was afforded throughout the length of the Baldovan Road trap, disturbed surface samples could be taken along the full length of the trap. These samples were generally taken at three points in the trap:

- 0.75 m downstream from the trap inlet;
- at the trap's longitudinal mid-point;
- 0.75 m from the trap outlet.

Pipe deposits were not always available for sampling in the immediate vicinity of the trap. In general, the deposits sampled at this location were more temporally and spatially consistent than those sampled at the Constable Street site. This is believed to be a result of the smaller range of particle sizes contributing to flows at this point (smaller catchment area) and more significantly, the reduced proportion of smaller sized particles. Fine particles were only detected during the initial stages of trap filling, with an apparent preference for these particles to deposit at the downstream end of the trap. This is a logical distribution, as these particles will have the greatest mobility and will therefore be carried further into the trap. As the trap filled, the trapped particle's characteristics again tended toward those of the limited upstream pipe deposits. Significantly higher than expected volatile contents were sampled during the early stages of trap filling, as very low rates of transport of fine, organic material were detected when suspended and bed load material were sampled. It can therefore be concluded that during these early stages, the trap is very efficient in collecting all particle sizes and is therefore not as selective as previously believed.

Day	U/S Pipe d_{50} (mm)	0.75 m d_{50} (mm)	1.6 m d_{50} (mm)	2.45 m d_{50} (mm)
3	1.03*	0.65	0.38	0.25*
46	-	0.85	0.72	0.45
129	1.19	1.28	1.32	1.15

Table 3.18 - Particle sizes for Baldovan Rd - Claverhouse Fill Pattern 2

* Note - only small sample could be obtained

Day	U/S Pipe % volatile	0.75 m % volatile	1.6 m % volatile	2.45 m % volatile
3	8*	79	92	94*
46	-	29	41	53
129	6	6	10	10

Table 3.19 – Volatile content for Baldovan Rd - Claverhouse Fill Pattern 2

* Note - only small sample could be obtained

Day	U/S Pipe ρ_b (kg/m^3)	0.75 m ρ_b (kg/m^3)	1.6 m ρ_b (kg/m^3)	2.45 m ρ_b (kg/m^3)
3	2022*	1299	1153	1088*
46	-	1622	1264	1234
129	2001	1965	1953	1955

Table 3.20 – Bulk density for Baldovan Rd - Claverhouse Fill Pattern 2

* Note - only small sample could be obtained

3.6.2.1.3 Forfar 900 Trapped Sediments

The surface sampling of trapped sediments at the Forfar 900 site offered a number of problems. As the trap tended to fill rapidly and the sediment/water interface was more of a graduated zone rather than a defined boundary, the actual extraction of samples at fixed points in the trap was difficult. Also, as a result of the high volatile content of the samples extracted from the site, dry sieving did not always produce quantifiable results. Consequently, as an alternative to the approach adopted for the other two sites, as many samples as possible were extracted from the trap in order to try and characterise the solids with a degree of confidence. This exercise revealed a wide potential variation in trapped sediment characteristics depending upon antecedent conditions and the potential dominance of any grits sampled in highly organic sample volumes.

Average particle sizes were found to vary from less than 63 μm to more than 1.2 mm. However as a result of the high organic content of a number of the samples, a meaningful particle size envelope cannot be prepared from the data, as in these cases up to 75% of material passed through all sieve sizes.

As a general rule, larger more granular material was extracted from the upstream end of the trap. This was particularly evident following heavy rainfall, with an armour of coarse grits often laid down on the previously trapped material. It was therefore hypothesised that significant stratification and consolidation of sediments was possible at the Forfar site.

Location	Max d50 (mm)	Min d50 (mm)	Max volatile solids %	Min volatile solids %	Max bulk density (kg/m3)	Min bulk density (kg/m3)
Pipe	2.2	0.25	82	1	2288	1824
U/S Trap	1.18	0.15	84	2	2301	1403
D/S Trap	0.8	<0.063	100	2.5	2059	1321

Table 3.21 - Physical characteristics of trapped and pipe sediments at Forfar 900 sediment trap

As a result of the concerns that the effects of sediments may have on treatment processes, the polluting potential of the Forfar sediments was assessed. The material was found to have a high polluting potential, with BOD_5 results shown in Table 3.22.

Location	Max BOD (mg/l)	Min BOD (mg/l)
Pipe	8252	461
U/S Trap	16365	1895
D/S Trap	22145	3111

Table 3.22 - Recorded BOD_5 concentrations for Forfar 900 sediment samples

3.6.2.2 Cryogenically Cored Samples

As the surface samples revealed a high temporal variability (as demonstrated by the Forfar trap armouring), it was decided to investigate if this variability could be logged through the extraction of a cryogenic core.

Cored samples were taken from all sites, with particular attention paid to the Forfar trap as it became apparent that the trap would be modified during the project in an attempt to enhance its poor performance. The cores were extracted once the traps showed no further increases in retained volume.

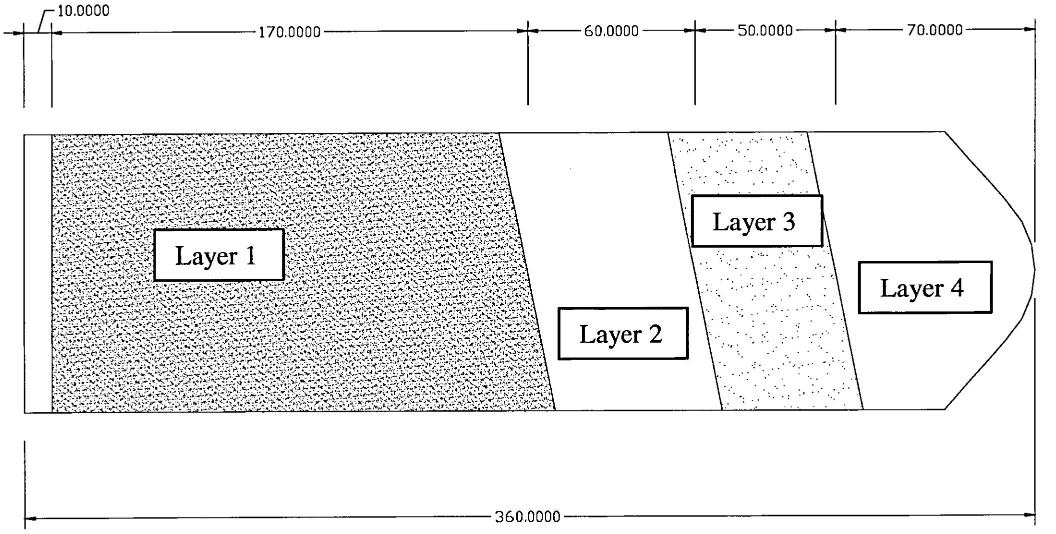


Figure 3.65 - Forfar sediment core 21/1/00

Figure 3.65 shows a diagrammatic representation of a core extracted from the Forfar 900 sediment trap on 21/1/00. The upper surface of the core is located on the left-hand side of the figure. This core was extracted following a change in the ambient hydraulic conditions of the site and was taken in order to gauge if this had an effect on the type of material settled in the trap. This is discussed further in Section 3.6.7. A laminated deposit structure was clearly evident, with the material at the top of the core dark and granular in appearance. This was followed by a light brown band of organic material, then a further band of dark grit before a final layer of light brown organics. The table below summarises the characteristics of these layers.

As can be seen, the upper most layer is significantly more granular and dense than the other layers. Its relative depth also suggests that these characteristics are associated with the new dry weather conditions rather than a previous storm. Layers 2 and 4 show characteristics in accordance with the previous dry weather deposits found at the site. Layer 3 separates these dry weather layers and corresponds to a storm event recorded during the trap filling data collection period.

Layer	Bulk Density (kg/m^3)	*d_{50} (mm)	Volatile Solids (%)
1	1662	0.2	3
2	893	0.1	57
3	1224	0.15	21
4	815	0.15	52

*N.B. – The d50 refers to the mineral material present only

Table 3.23 - Summary of physical characteristics of Forfar 900 core 21/1/00

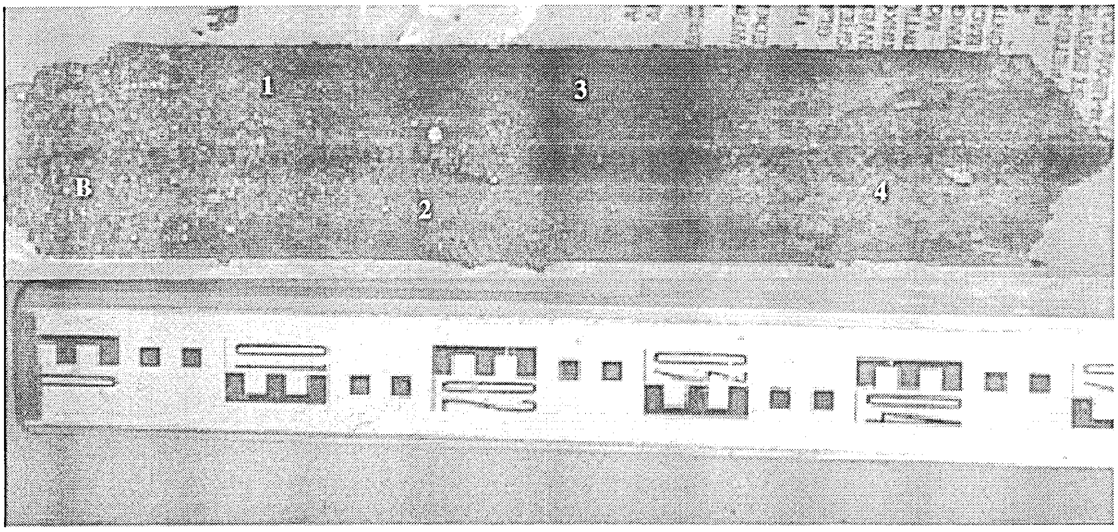


Figure 3.67 - Forfar sediment core 30/5/00

Figure 3.67 (above) shows a photograph of a core extracted following the third fill pattern of the Forfar 900 sediment trap, with layer B (left) denoting the top of the cored sample. In this case, although more grits were found in general throughout the core, a dark layer of grit material is evident in layer 3. The upper-most layer is labelled B as this is believed to contain the material in bed transport at the time of sampling. Layer 1 was found to be a thin layer of a mineral deposit dominated mixture of sediments. Layer 2 was found to be a more balanced mixture of organic and inorganic material, with Layer 3 clearly predominantly inorganic. The bottom-most layer was observed to appear similar in structure to layer 2. Table 3.24 summarises the physical characteristics of these sediments.

Layer	Bulk Density (kg/m^3)	*d_{50} (mm)	Volatile Solids (%)
B	1001	0.2	78
1	1360	0.3	15
2	954	0.32	31
3	1392	0.4	24
4	1113	0.35	33

*N.B. – The d50 refers to the mineral material present only

Table 3.24 - Summary of physical characteristics of Forfar 900 core 30/5/00

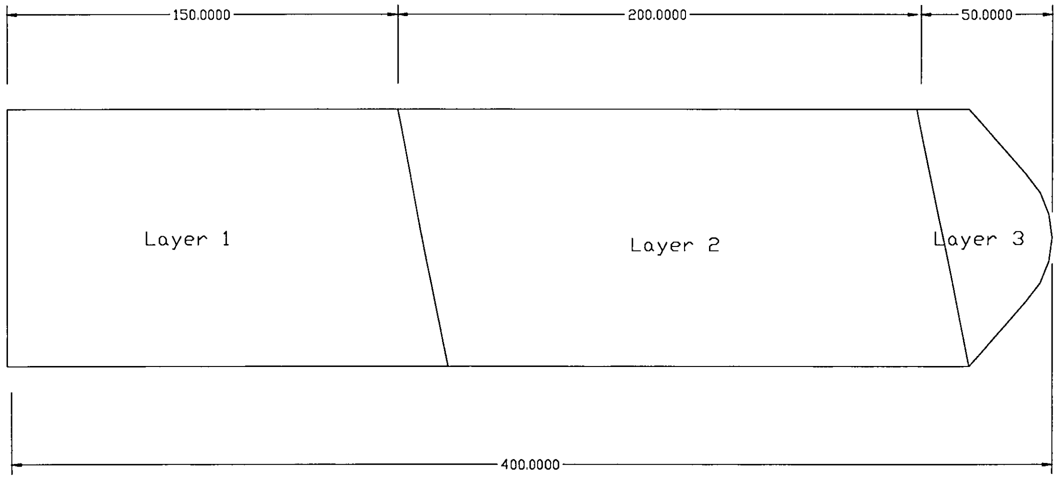


Figure 3.69 – Baldovan Rd - Claverhouse sediment core 16/2/00

Figure 3.69 (above) represents a core taken from the Baldovan Road - Claverhouse site following trap filling, with layer 1 representing the uppermost layer. Laminations at this site were much harder to distinguish as the characteristics were observed to change gradually from top to bottom. It was evident that the sample exhibited almost purely mineral characteristics at the sample surface, with a mixture of organic and inorganic material found at the core base. The sample was therefore divided in order to characterise these changes. Table 3.25 summarises the physical characteristics found. It can clearly be seen that when the trap is empty, a greater proportion of organic material is retained (as in-trap velocities are significantly lower than those in the local pipes). However, as the trap fills (and trap velocities increase as a result of geometry changes), lighter organic matter is carried over the trap. These results closely match the surface samples taken from the site during trap filling. This close match is believed to result from the lower variability of sediment inputs observed at this site and the dominance of inorganic material at all times.

Layer	Bulk Density (kg/m^3)	*d_{50} (mm)	Volatile Solids (%)
1	1950	1.05	8
2	1245	1.12	41
3	1141	0.3	94

*N.B. – The d50 refers to the mineral material present only

Table 3.25 - Summary of physical characteristics of Baldovan Rd - Claverhouse core 16/2/00

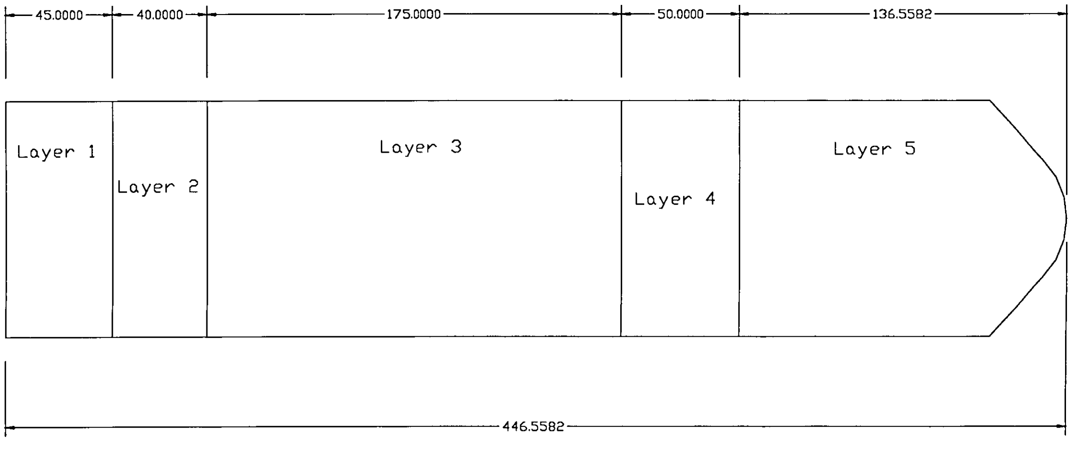


Figure 3.71 - Constable Street core 11/5/00

Figure 3.71 (above) shows the large number of layers found in a core extracted from the Constable Street site near the trap inlet, with the top of the core to the left (layer 1). The uppermost layer was found to be relatively organic in nature and is believed to arise from recent dry weather solids loadings. This was followed by a very granular layer corresponding with a period of intense storms approximately 4 days before the sample was taken. Layer 3 contained a mixture of organic and inorganic material and overlaid another predominantly inorganic layer. Layer 5 was observed to exhibit characteristics similar to those of layer 3. Table 3.26 summarises the physical characteristics of these layers. It is believed that the general large size of the inorganic particles is a consequence of the sampling location (at the trap inlet). At this location the largest, most dense particles will be deposited preferentially.

Layer	Bulk Density (kg/m^3)	*d_{50} (mm)	Volatile Solids (%)
1	1130	0.4	82
2	1526	2.4	9
3	1233	0.8	32
4	1403	1.5	8
5	1251	0.9	48

*N.B. – The d50 refers to the mineral material present only

Table 3.26 - Summary of physical characteristics of Forfar 900 core 30/5/00

3.6.3 Bed-Load Transport Rates

Where possible, bedload transport rates were measured directly using the miniature bed-load traps described in Section 3.5.4.2.

3.6.3.1 Constable Street Bed-load Transport Rates

The bed traps were installed a number of times in order to attempt to build up a picture of the diurnal variation in transport rates. The day was divided into 6 – four hour periods (7am to 11am; 11am to 3pm; 3pm to 7pm; 7pm to 11pm; 11pm to 3 am; 3am to 7am). These times were chosen in order to try to differentiate between peak and low flows. Significant bed-load transport was only detected at peak flows with no material collected between 11pm and 7 am. Figure 3.73 shows the average rates calculated from three sets of bed trap data.

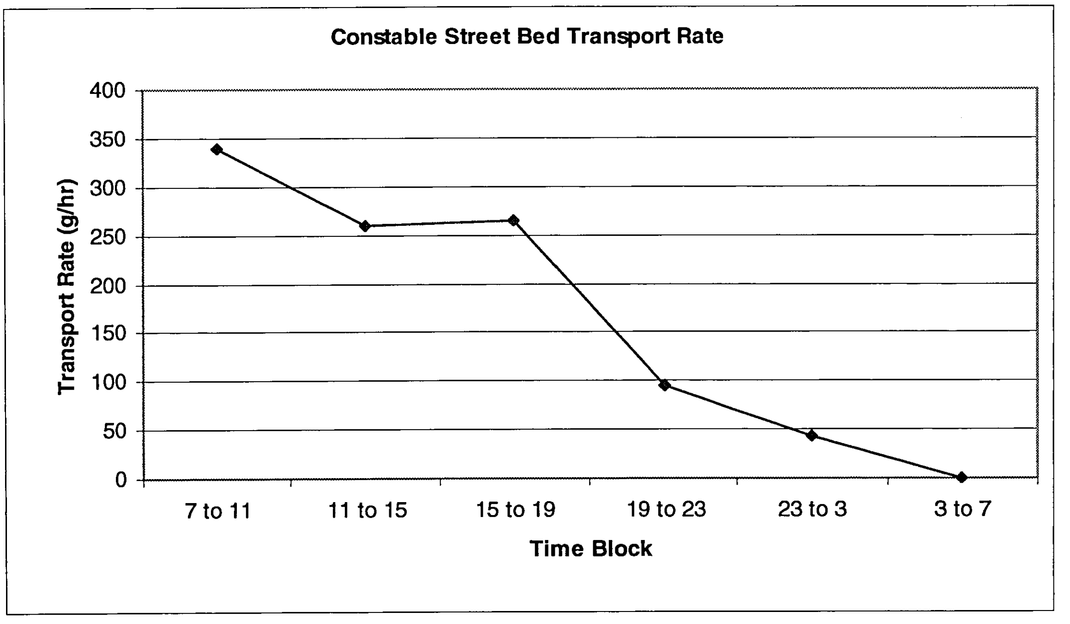


Figure 3.73 – Averaged Constable Street DWF bed transport rates (3 day average)

3.6.3.2 Baldovan Road - Claverhouse Bed-load Transport Rates

A similar procedure was attempted at the Claverhouse site. However, no analysable samples could be collected within the 4-hour time slots. It was found that a reasonably sized sample could only be collected when the traps were left in-situ for a period of 24 hours. In this way, an average bed transport rate of less than 12kg/day (approximately 0.01 m^3/day) was determined. This is significantly lower than the rates experienced at the other sites and is also lower than the fill rates observed during the early stages of trap filling at this site. These anomalies are the result of a number of factors:

- The low number of foul inputs located upstream from the trap;
- The changes to the overall hydraulics as a result of collecting these data at a time when the trap was partially filled;
- The tests were carried out during dry weather which is not comparable to the mixed storm/dry conditions used to derive the fill rate of the trap:
- Uncertainty of the performance of the small scale traps under the hydraulic conditions at the Claverhouse site (comparatively fast shallow flows).

It should however be noted that these data compare favourably in relative terms with the overall fill rates observed when the main trap is partially filled (approximately – 35% error).

3.6.3.3 Forfar Bed-load Transport Rates

The bed transport rates found at the Forfar 900 site were found to be difficult to collect as a result of their nature (highly mobile bed) and high solids loadings. It was not always possible to ascertain the source of the material in the traps as the action of opening the lid of the bed traps was observed to disturb both the upstream bed material and mobile solids within the trap itself. As would therefore be expected, a high degree of variability in near bed transport rates was found to exist at the site, with each of the data sets providing a range of approximately +120% to –45% for any given time slot. It was also difficult to determine the point at which the trap became full as a result of the turbidity of flows. It is estimated that the miniature traps used to establish the bed transport rate essentially became full after only 15 minutes.

Figure 3.75 shows the averaged data from the Forfar miniature bed traps. It is clear from this that the diurnal profile observed at other sites does not occur here. The transport rates are observed to remain relatively constant throughout the day with a reduction only observed in the early hours of the morning.

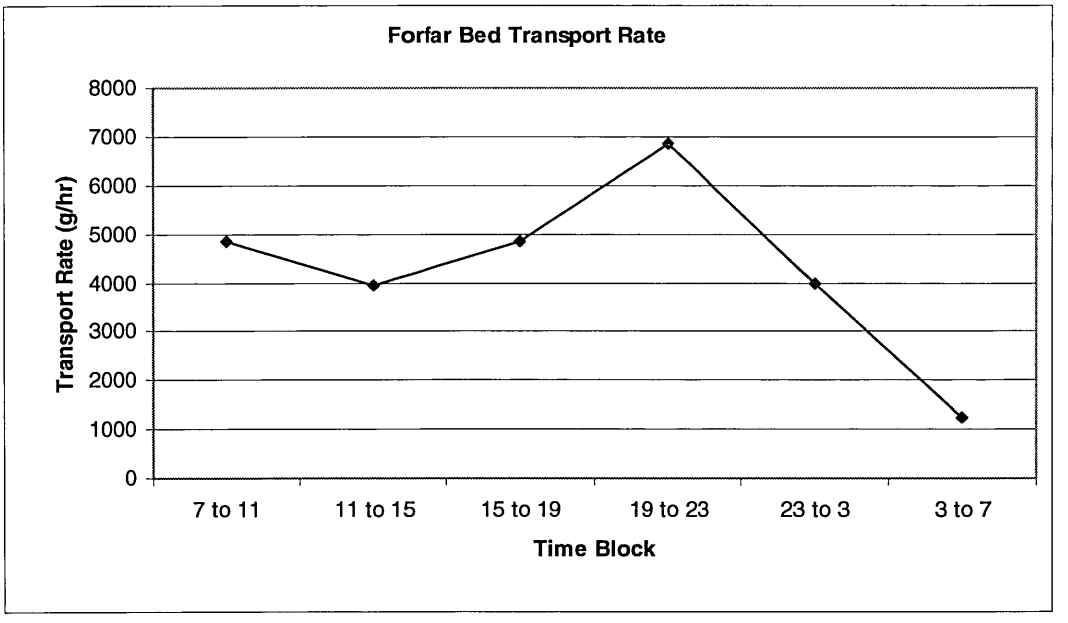


Figure 3.75 – Averaged Forfar 900 DWF bed transport rates (3 day average)

3.6.4 Suspended-Load Transport Rates

Dry weather flow suspended solids transport rates were established over a 24-hour period at each site. Multi-depth sampling was used where possible to allow the sediment load to be accurately determined. The samples were then filtered to allow the total suspended solids (TSS) content of each sample to be determined.

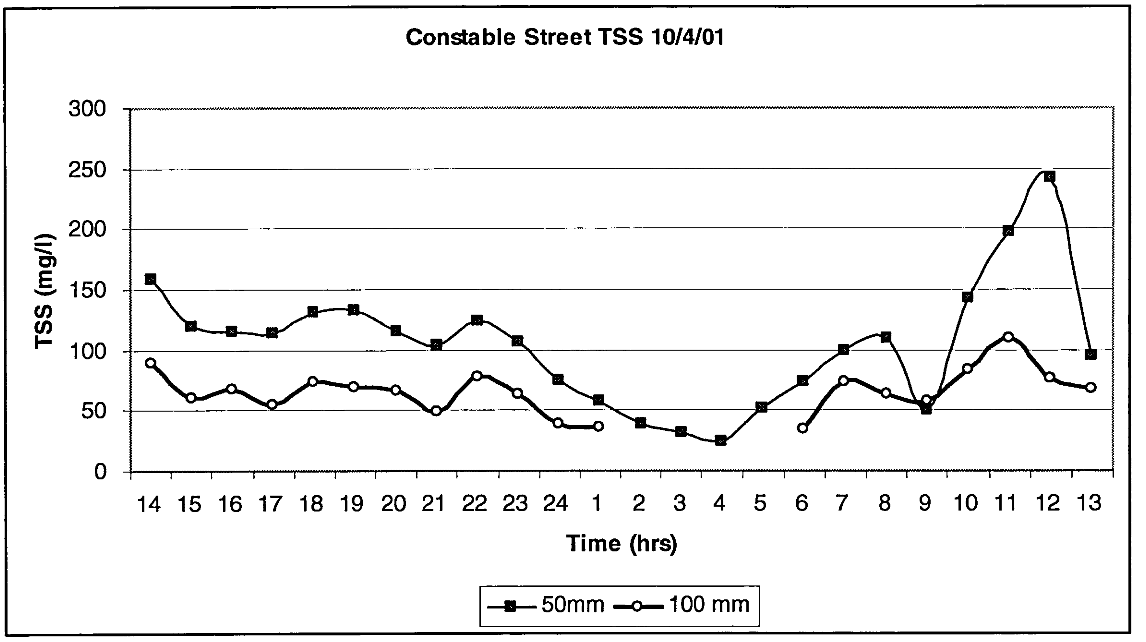


Figure 3.77 - Constable Street DWF suspended solids concentrations 10/4/01

Figure 3.77 shows a typical profile obtained from the Constable Street trap inlet, with plots shown for a sampling tubes located 50mm and 100mm above the sediment bed. Only two depths are sampled at this location as a result of the low flow depths experienced at the site despite the flows being in the order of 60 l/s. The effect of these low depths is highlighted during low flows as the uppermost sampling point (100mm above bed) is unable to collect a sample at these times. Throughout the sampling period a distinct profile was detected, with higher concentrations observed nearer the pipe invert (50mm above bed).

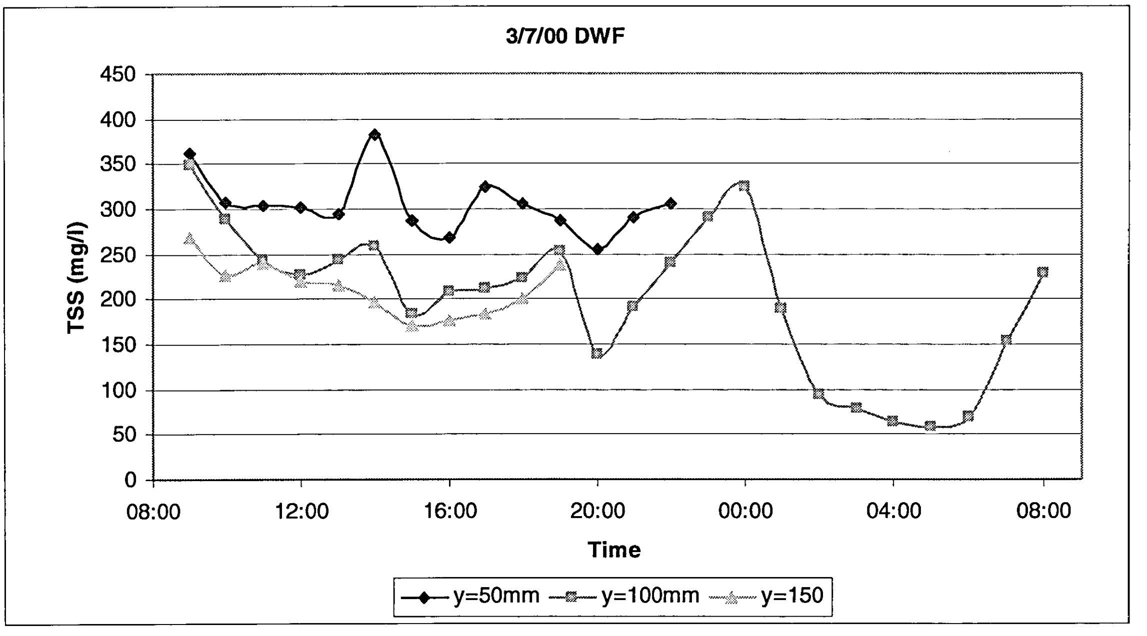


Figure 3.79 - Forfar DWF suspended solids concentrations Day 1

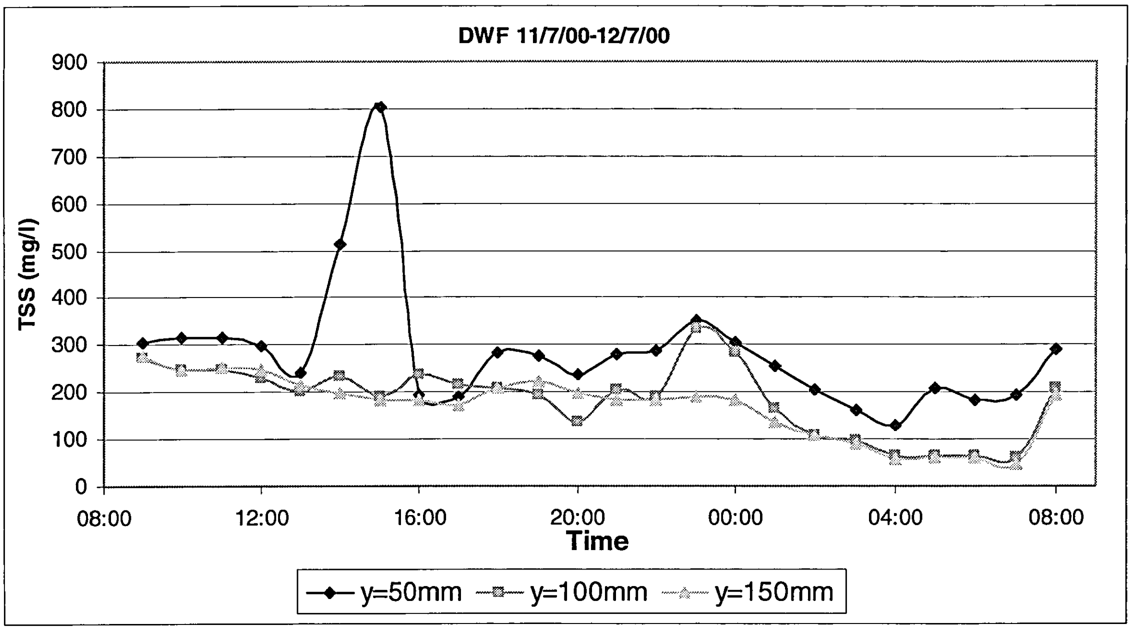


Figure 3.81 - Forfar DWF suspended solids concentrations Day 2

The deeper flows of the Forfar site allowed three sampling points to be used. Although the flows were comparatively tranquil at the time of sampling, a distinct increase in sediment concentration with increasing proximity to the sediment bed was detected. Sediment loadings were observed to be generally higher at the Forfar site than at any other sampling location.

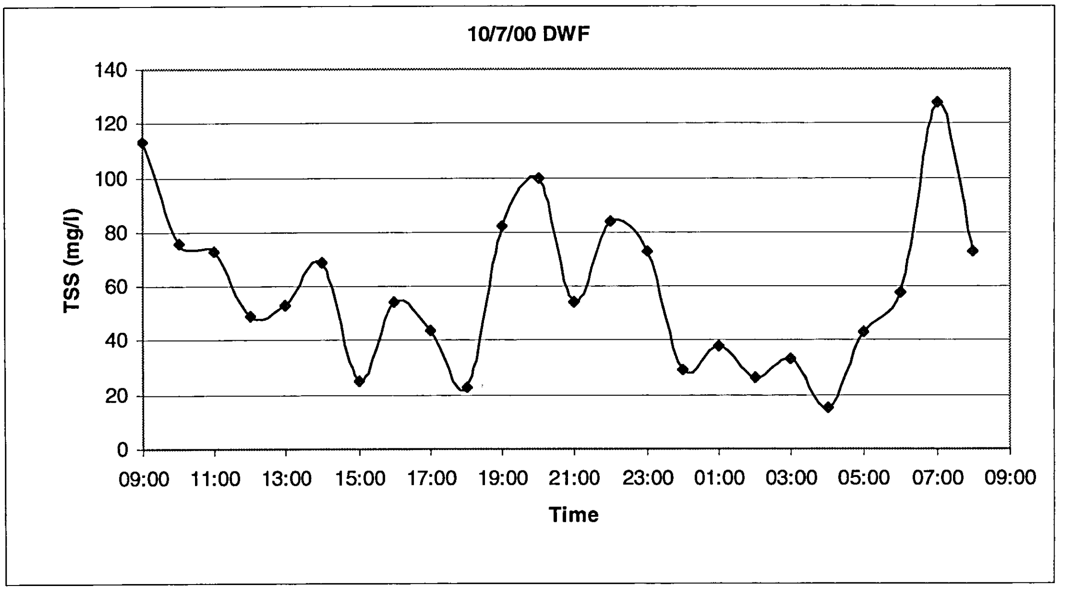


Figure 3.83 – Baldovan Road- Claverhouse DWF suspended solids concentrations 10/7/00

As a consequence of the low flow depths experienced at the Baldovan Road - Claverhouse site, only 1 sampling point could be used, at a height of 35 mm above the pipe invert. Only one set of complete data could be collected as even at this single low depth, sampling was often found to be affected by the low flows. Figure 3.83 shows the resulting diurnal variation of TSS. The loadings and hydraulic flows were observed to be highly variable at this site with pulsed flows often detected. This is believed to be a result of the significance of the input of a small hospital located upstream from the site. The concentrations observed at his site were the lowest of all the sites monitored with concentrations frequently below 40 mg/l.

3.6.5 Pipe Deposit Levels

Historically (and during part of this programme of research) the hydraulics of the Murraygate Interceptor Sewer have been partially restricted as a result of a downstream control. This control took the form of cemented "permanent" sediment deposits. The investigations carried out within this study revealed that the level of sedimentation in the Murraygate sewer was far in excess of that which should exist under free flowing conditions. The development of the sediment deposition models suggested an equilibrium sediment level in the order of 50 mm as opposed to the 150 mm frequently observed within this important sewer length. The decision was taken by the Water Authority to remove the hydraulic control in an attempt to reduce the risk of surcharge in the pipe and reduce cleaning costs as a result of a regular maintenance programme required to limit the deposition.

Following the removal of the downstream control, regular "walk-through" surveys were undertaken, recording in detail the changing sediment depths, bed gradients and sediment quality. This data set was then used in the validation of the sediment deposition and erosion models developed for this study using individual events where possible in addition to the prolonged data set.

Figure 3.85 shows how the sediment levels within the Murraygate sewer reduce following the removal of the downstream control. The invert level of the sewer along this length is shown as the bold black line. The invert level can be seen to be irregular over the first 60 m of the survey length. Following this length, the sewer gradient becomes more regular. This invert irregularity can be seen to have a significant influence on the patterns of sedimentation. Sediment levels are shown using the remaining graph lines, with each coloured line representing the detailed profile of sediment depths recorded on each individual walk-through date. In the downstream length, a large reduction in overall sediment depth is experienced during the survey. This is most clearly seen when comparing the first survey, shown in yellow (24/04/00), with the last (13/03/01) shown in black. Occasional high spots in sediment level are recorded in all walkthroughs. It is believed that these result from the movement of storm solids, pulsing through the system. However, sediment levels in the upstream portion are more constant with only a minor reduction observed. The reason for this is that the full improvement to hydraulics is not felt at the inlet to the test length as the invert irregularity exerts its own hydraulic control. This length will therefore not be freely discharging unless work is undertaken to reconstruct a more regular invert surface.

It was believed that the sediments would erode initially from downstream locations, with this pattern gradually spreading upstream. This behaviour was not as pronounced as expected with only a marginal preference for downstream erosion observed. For the purposes of modelling data, only the lower, freely discharging section was considered.

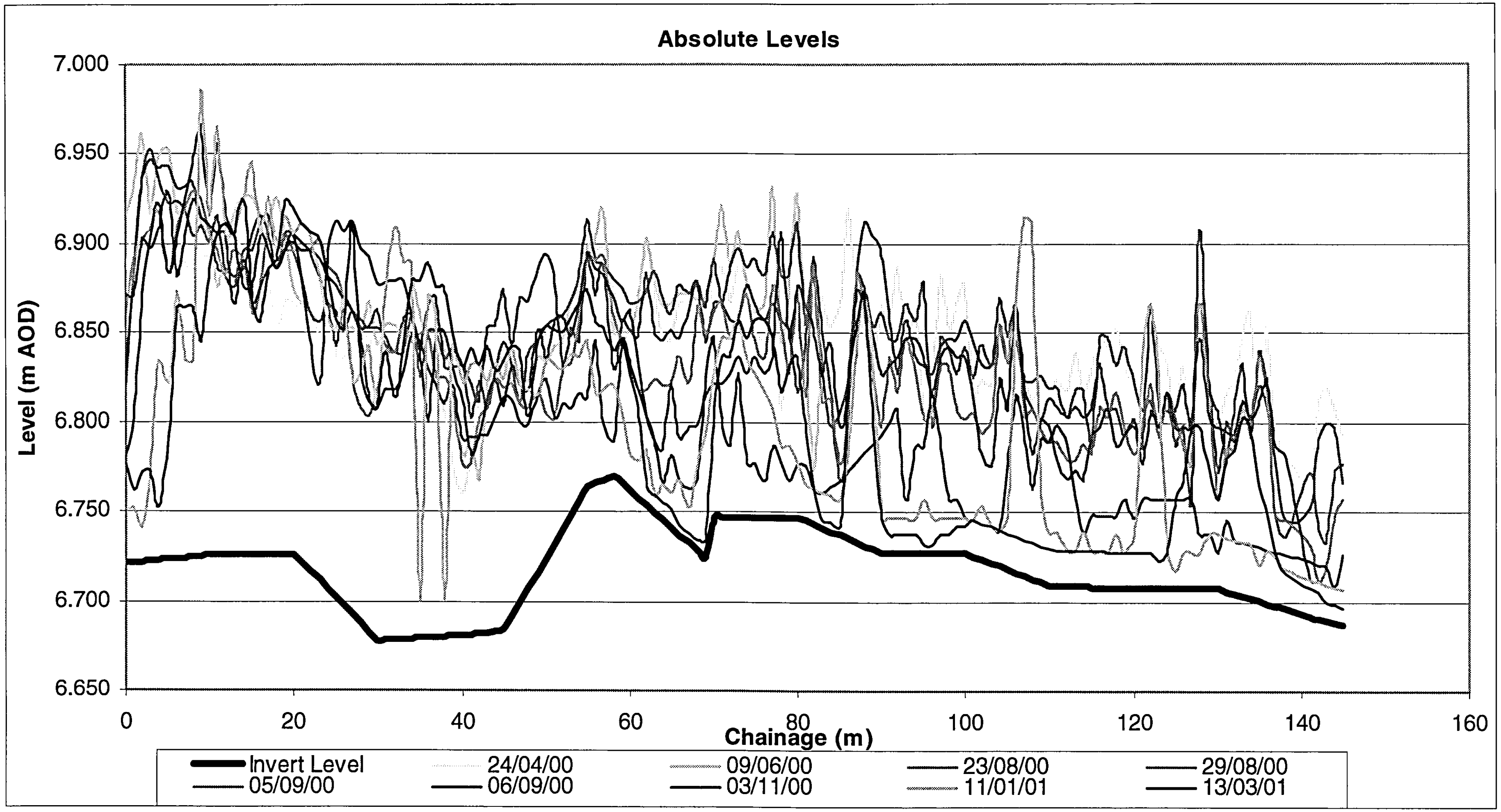


Figure 3.85 - Reducing bed levels following removal of downstream control

3.6.6 Pipe Deposit Characteristics

At fixed points along the length of the survey, samples were extracted to determine the physical characteristics of the deposits as the new bed profile evolved. Although there was a marginal increase in the density and diameter of particles during the survey period, this was not found to be statistically significant. The average characteristics at each point are therefore provided in Table 3.27.

	0m	40m	83m	120m	135m	140m
Bulk Density (kg/m^3)	1859.3	1727.2	1827.2	1682.65	1952.55	1608.55
M C (%)	16.26	24.43	26.27	34.68	21.70	22.55
d_{50} (mm)	0.425	0.320	0.515	0.435	0.415	0.420
Liquid Content (%)	19.42	32.32	35.62	53.09	27.72	29.11
Volatile Solids (%)	2.15	13.00	4.05	8.19	17.35	12.09
Dry Density (kg/m^3)	1556.95	1305.30	1347.25	1099.15	1528.75	1245.90

Table 3.27 - Average characteristics of Murraygate sediment deposits

3.6.7 Forfar 900 Trap Modifications

It was clear from the data collected at the Forfar trap that the trap was not operating as was originally intended. The trap was filling far too quickly and with highly organic, low-density material that is best dealt with at the nearby treatment plant. As continuing operational problems were being experienced both in the network and treatment plant, a request was made by North of Scotland Water Authority to investigate the causes and to suggest potential improvements to the existing trap arrangement.

Forfar treatment plant and pump overflows are located immediately adjacent to the potentially eutrophic Forfar Loch. As a consequence of this, the environmental regulator (SEPA) has issued strict discharge consents with the proviso that a sufficient level of in-sewer storage should be maintained in the incoming trunk sewers. The level of deposition in these sewers clearly restricts the level of storage available. It is therefore necessary from this viewpoint to reduce the level of sedimentation. In addition to this, the potential for storm flushes was investigated through the installation of flow and sampling equipment at the inlet to the works. During a four-week survey period, two significant rainfall events were noted to occur. Few quality data were collected during the first event as a result of wastewater sampler failure. However, an almost complete data set was returned for the second event in order to give an indication of the flush effects present in the Forfar system.

Figure 3.87 and Figure 3.89 show the results of this data collection activity. As can be seen, a significant total suspended solids flush follows the general pattern of the storm's hydrograph, with only a relatively minor dilution observed during the storm tail. This suggests that there is sufficient bed material within the system to provide solids for erosion for the entire event.

The COD profile generally mirrors that of the TSS, as has been generally found in other studies. However, analysis of the NH_4 concentrations reveals a distinct ammonia flush, prior to the more usual dilution of ammonia concentrations. This peak is believed to arise from the significant volumes of interstitial liquid held within the highly cohesive but readily eroded near bed solids observed in the Forfar trunk sewer. This suggests that the type of flush observed will depend on the mode of sediment transport and nature of sediment deposits located within a system.

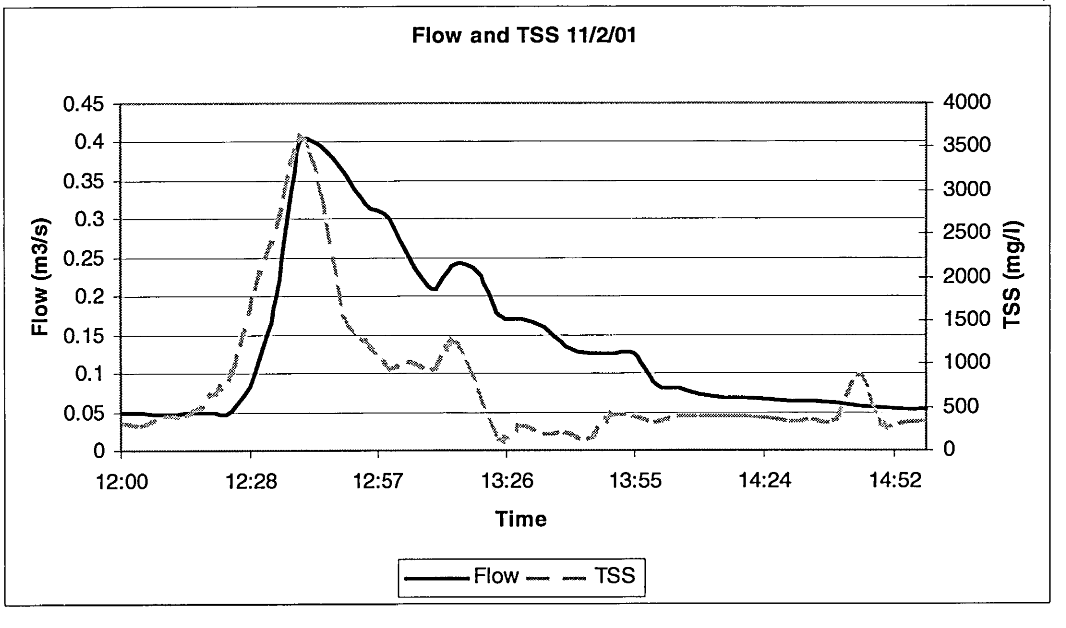


Figure 3.87 – Flow and TSS Forfar storm event data - 11/2/01

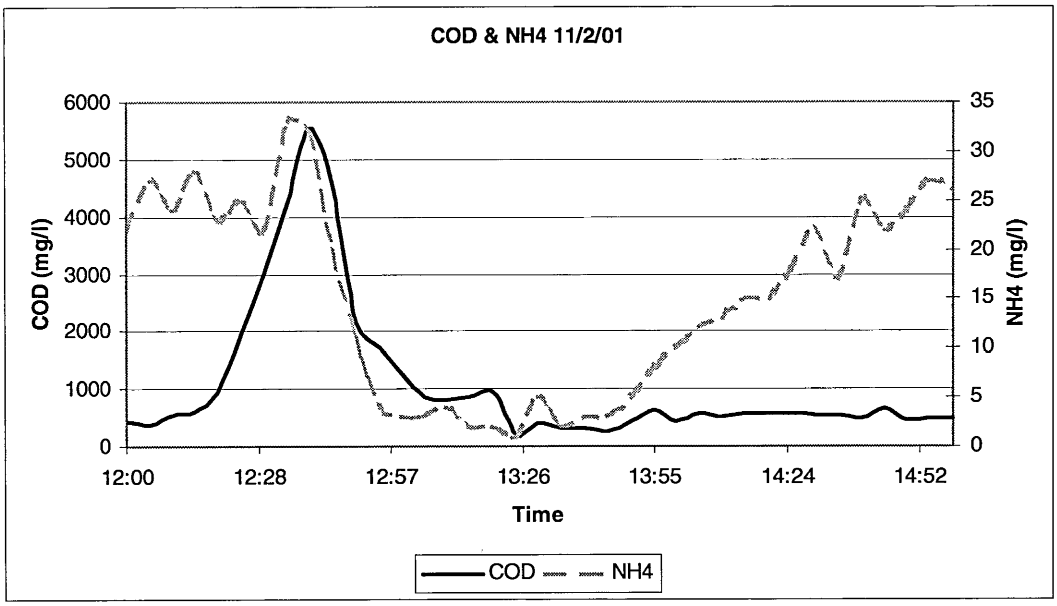


Figure 3.89 - COD & NH4 Forfar flush event 11/2/01

The initial assessment of the system confirmed the concerns of the Water Authority regarding the potential for large flushes and storage and capacity restriction. Following observation of the filling of the existing traps and the collection of sediment samples (both in transport and bed material as detailed in the preceding sections) it was clear that the hydraulic regime was not sufficient to allow the selective settlement of problematic deposits. This is principally a result of the location chosen by the Water Authority to install the traps. The traps were installed at the location of significant sediment deposits in order to provide more storage and it was believed that as this was a location where material settled, the traps would operate effectively. However, the traps should have been installed at an upstream location in order to intercept the more readily settleable material before it is allowed to deposit and further influence the hydraulics and thus encourage other particles to settle.

As part of this study, HydroWorks modelling was carried out in order to assess if the pumping regime could be improved to reduce sedimentation. The modelling work suggested that more frequent pumping should be carried out. Unfortunately the configuration of the pumps meant that the rate of pumping could not be adjusted.

A programme of root cutting was recommended for the downstream sewers and was carried out by the Water Authority in order to try and improve the hydraulic regime. In addition to this, checks were made to ensure the frequent operation of the pumps at the inlet to the treatment works as these were found to have a significant effect on the hydraulics of the trunk sewers. The consequential sediment levels were then recorded at regular intervals and a steady reduction in sediment bed depth was observed. This reduction is shown in Figure 3.91. Over the period of surveys, a reduction in average bed depth of 200 mm was observed. This dramatic reduction in sediment depth was brought about through an increase in bed shear from less than 0.1 N/m^2 to 0.75 N/m^2 as a result of the root cutting and pump alterations. This resulted in coarser deposits being found in the vicinity of the sediment trap as the increased shear forces selectively entrained the fine material and transported it to the WWTP. Figure 3.93 summarises the sampling data taken from a core extracted following the hydraulic improvements. The material may be compared to the trapped sediments as samples taken prior to the hydraulic change indicated that trapped and pipe samples were not significantly different.

The changes in the trunk sewers were also reflected by observations at the treatment plant, with increased grit arriving at the plant and a reduction of large pollutant loadings during storms.

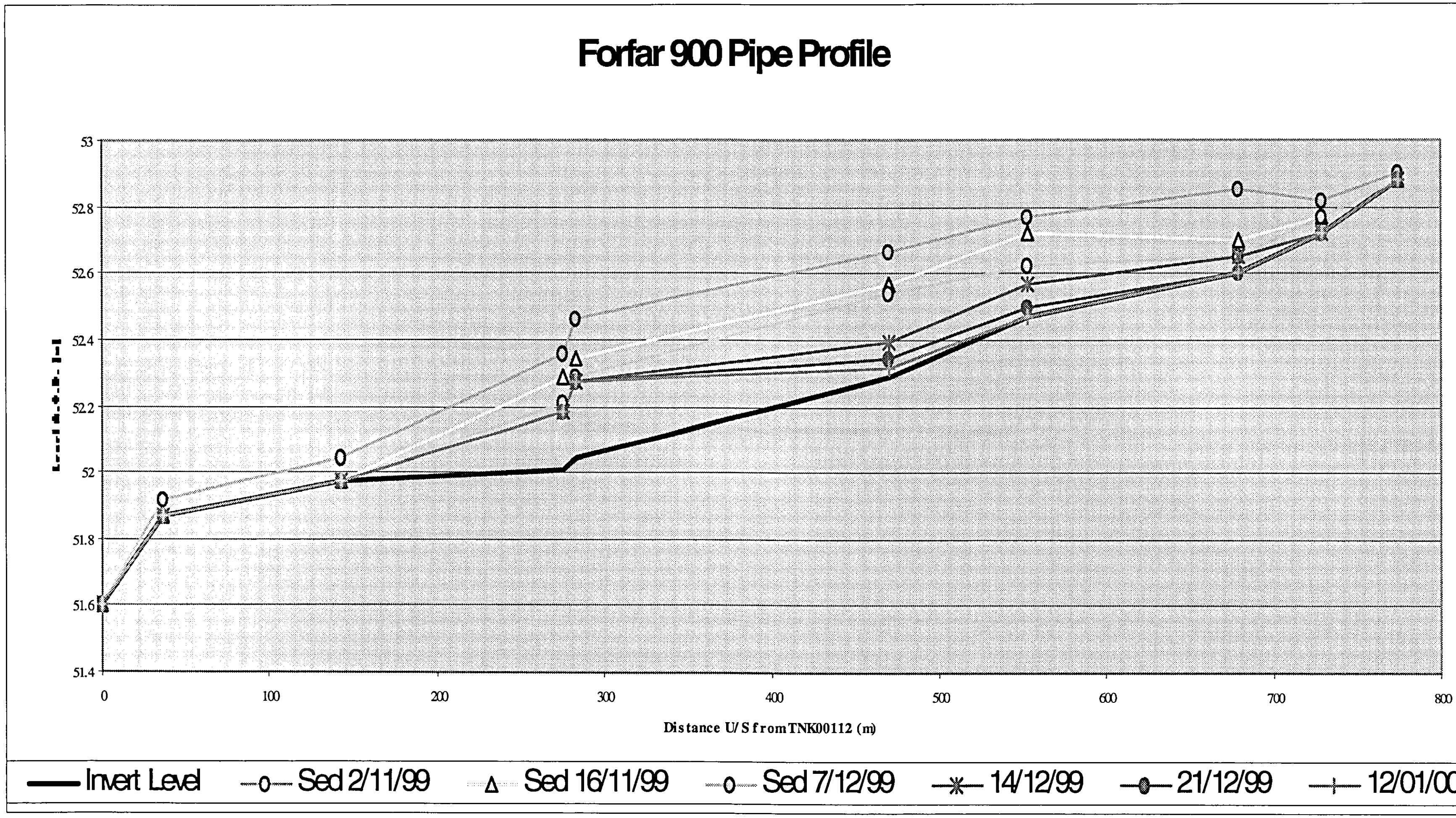


Figure 3.91 - Forfar 900 trunk sewer sediment levels following hydraulic improvements

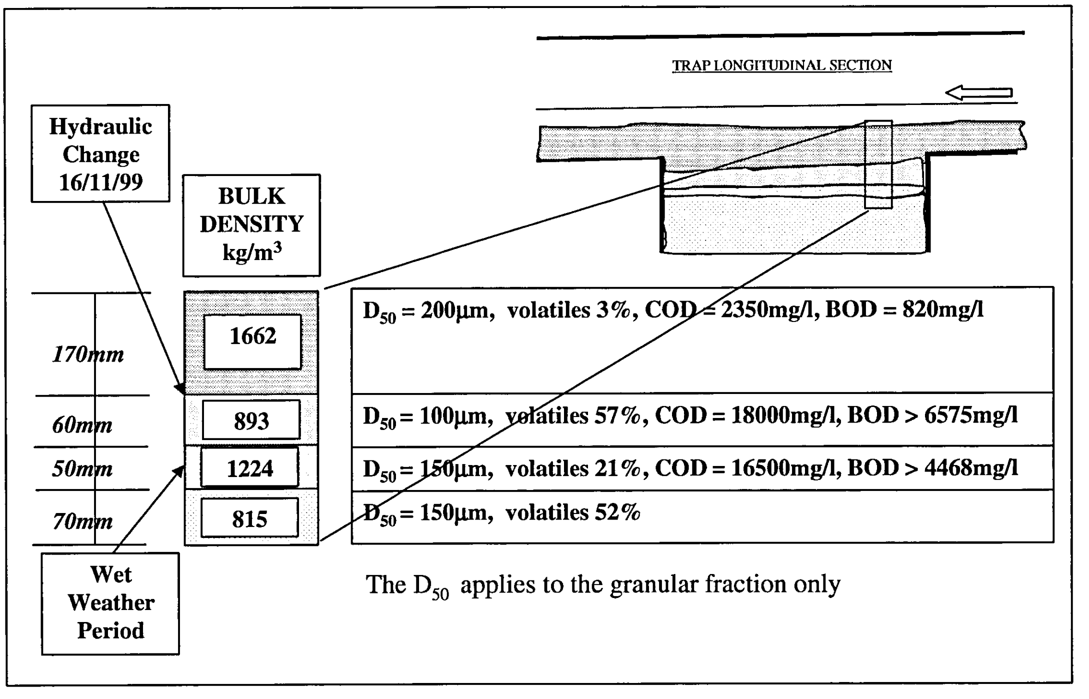


Figure 3.93 - changes observed in sediment deposits

In addition to these operational improvements, it was decided to attempt to use an improved trap design along with the improved hydraulics in an attempt to selectively trap larger mineral material at the existing trap location. The data collected earlier in the study were used to drive some CFD modelling being carried out as part of the parallel studies at the University of Sheffield. The CFD studies were at this stage embryonic, but were advanced sufficiently to allow a coarse estimate of various designs for testing. The CFD analysis suggested that the most appropriate configuration was to partially cover the traps, with a central slot width of 300 mm. This design was selected on the initial assumption (supported by CFD analysis) that the use of the partial covers and slots would allow increased selectivity of trapped particles. This configuration was replicated in the actual trap through the construction of an aluminium frame supporting a pair of marine-ply grade wooden covers. Each of the covers contained a 600 mm wide hinged door in order to facilitate access for sediment scans or the extraction of samples.

As a result of knowledge of the fast previous rate of filling, a short interval between site visits was implemented in order to gain sufficient data. Consequently the first scan was scheduled for 64 hours after flows were reinstated to the trap. Even this short interval was found to be too long as the trap was found to be 80% full following this period. The trap was scanned again a further 2 days later and was found to be full. It was hoped that during this stage that perhaps dense granular material would still enter the trap and displace the highly organic material found in the new trap configuration. This was not observed to happen.

Samples of the trapped material were extracted and compared to both previous trap and pipe samples. The samples extracted from the partially covered trap were found to be significantly more organic (approaching 100%) and with a lower bulk density (1030 kg/m^3) than those previously found in either the trap or local pipes. Previously the trap had collected particles of broadly similar characteristics of those of the pipe. However, when using the partial covers in this particular location, the sediments were observed to be significantly different (more organic and of lower density). As the initial purpose of the partial covers was to preferentially select the granular fraction of transported sediments and hence increase the time taken to fill the trap, this is the exact antithesis of the intended effect of partial covers. This behaviour could not be replicated using the CFD modelling and was not expected. It is however hypothesised that as a result of the low velocities experienced at the site, the role of re-circulations within the trap becomes more significant to the overall hydraulics. There is therefore a tendency for low-density material to be effectively "sucked" in from above before coming to rest within the trap. The trap was emptied and this behaviour was again repeated.

This experiment provides valuable lessons regarding the placement and suitability of any particular trap design. The modifications were carried out in order to attempt to make the current trap locations operationally useable. However, although activities were carried out on a number of fronts, which improved the overall performance of the system, trap performance could not be enhanced. It can therefore be concluded that these traps are simply in the wrong location as the ambient hydraulics do not support the transport of even fine material of low density. This highlights the importance of ensuring a suitable hydraulic regime prior to the selection of a trapping site rather than locating traps at a convenient location or where sediment is known to be a problem.

The correct procedure at this location should have involved the work carried out as part of this study (root cutting and pump optimisation) rather than the installation of the sediment traps. Alternatively the installation of flushing gates could have been considered as these may have encouraged more of the fine material to arrive at the treatment plant downstream.

These field activities have been further used to develop general rules of the applicability of traps and the particular design that may be appropriate.

Chapter 4 : Development of Models

4.1 Introduction

The development of analytical tools for the prediction of sediment behaviour in sewers is central to the work carried out in this thesis. The assessment of previous studies and approaches has allowed the most suitable, flexible and robust methods to be identified. These approaches have then been enhanced by the creation of novel techniques to allow a sediment modelling procedure and software tool to be developed which enables deposition locations, sediment transport rates, deposition rates, erosion rates and the rates of sediment trap deposition to be determined.

This chapter describes the evolution of each individual model component, the application and testing of these components, and the development of a combined model mimicking the interactions of the various component parts.

4.2 Modelling Needs

At the outset of the study, the basic requirements of the modelling tools were defined. These needs were:

- To produce rapid, detailed, continuous hydraulic simulations with long-term durations (exceeding 1 year);
- To determine the most likely locations of sediment deposition using a pipe by pipe analysis;
- To predict depths of sedimentation at the locations identified as being at risk from sedimentation;
- To predict potential erosion masses and hence sediment concentrations during storm events;

- To provide sediment concentration inputs (suspended and bedload) to both the pipe deposition and trap models for dry weather and storm conditions;
- To determine the rate of filling for different trap types;
- To combine the individual techniques into a single modelling method.

The structure of this perceived model is shown below in Figure 4.1.

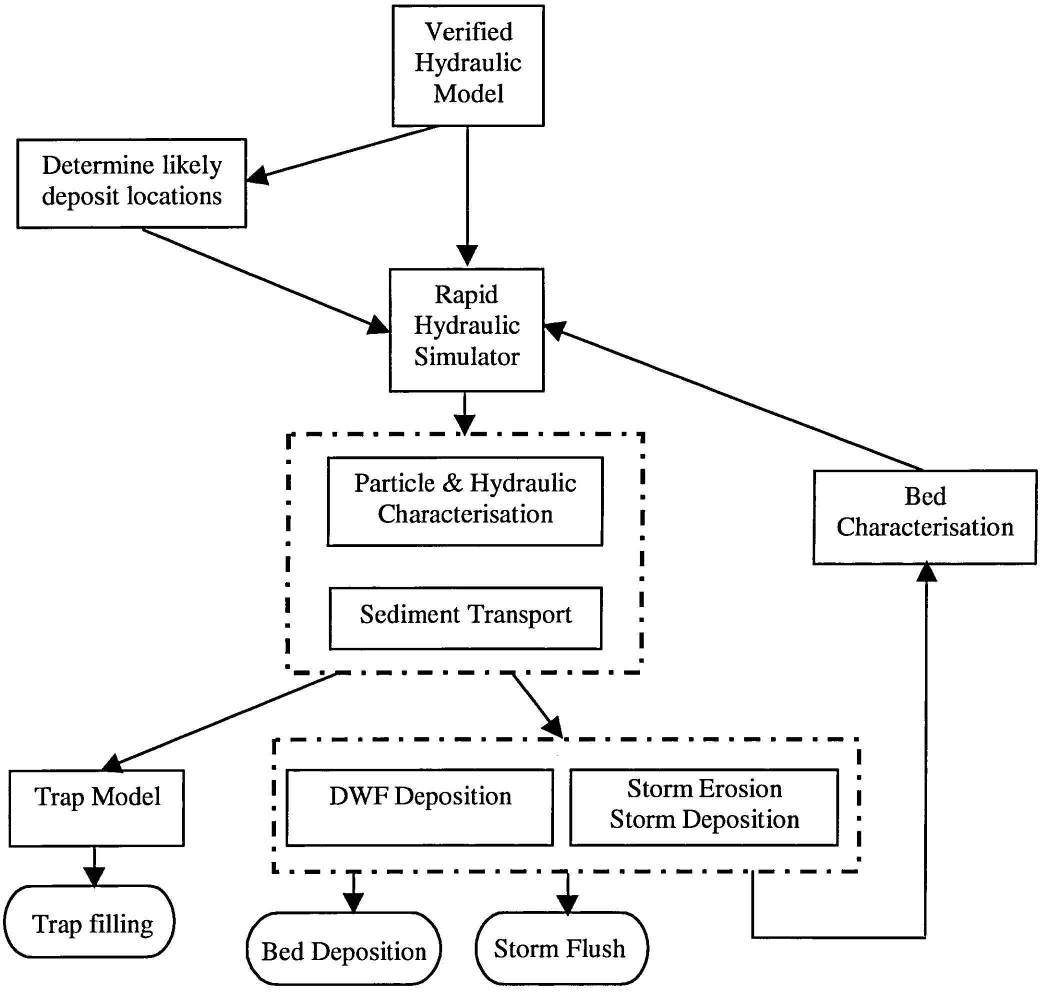


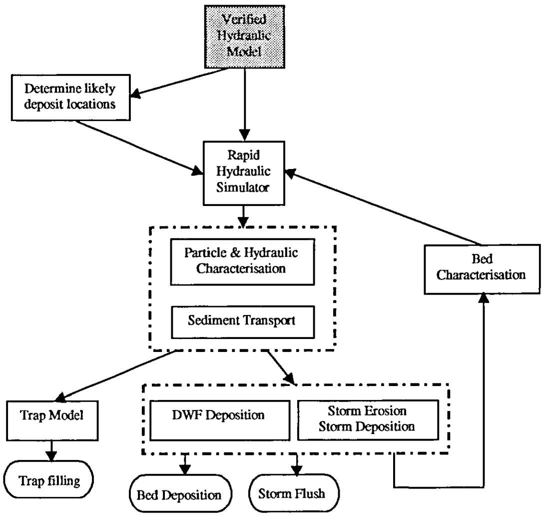
Figure 4.1 - Initial model schematic

The outset of the modelling process commences with the use of a verified hydraulic model for the catchment. This model is used to determine locations of probable sediment deposition through time series analysis and develop a rapid hydraulic simulator for these locations. The outputs of the hydraulic simulator are therefore flow, depth and velocity at points in the system where deposits have been identified either by operational experience or the use of the sediment location model.

These outputs are then used to drive the calculation of sediment erosion and transport rates, which are then combined with the hydraulic data to predict sediment deposition behaviour in pipes and sediment traps.

At each timestep, a feedback loop of information is proposed to allow the model components to interact. Each of these component model parts is discussed in more detail in the following sections.

4.3 Verified Hydraulic Model

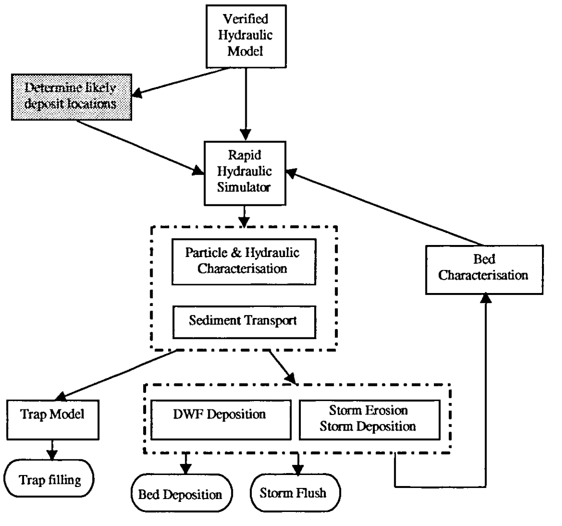


The verified hydraulic model will usually take the form an InfoWorks or HydroWorks model, verified to WaPUG Code of Practice standards (WaPUG, 2002).

This model can then be used for two principal functions:

1. As a detailed hydraulic input for the pipe by pipe sediment location model. This function is described further in Section 4.4;
2. As a benchmark for the calibration of a rapid hydraulic simulation model. This function is described further in Section 4.5.

4.4 Sediment Location Model



In order to facilitate a proactive sediment management strategy, knowledge of where sediments are likely to deposit is vital. This information can be used to schedule sensitive areas to be monitored and regularly cleaned, and if modelling facilities exist, flow regime changes to improve system performance may be attempted. In the case of the method used in this study, areas highlighted to be at a high risk from sediment deposition can then be modelled further to determine how great that risk is and devise a suitable solution.

In Chapter 2, it was revealed during the study of methods for the prediction of sediment deposit locations, that the approaches were generally either oversimplified, or contained a level of complexity not warranted by the inherent inaccuracies of estimating model inputs.

It was therefore proposed that any method selected would have to consider the complex processes involved in sedimentation in a simplified way. The WRc approach (Gent & Orman, 1991) uses a logical structure to assess sediment locations but is cumbersome to apply and limited, as it relies on look-up tables for simple cases (i.e. round clean pipes). The approach was described in detail in Section 2.8.1.

In essence, this unpublished method utilises detailed hydraulic modelling to produce flow depths and velocities throughout the entirety of the catchment. These are related to shear stress thresholds. Each stage of the method was reassessed using more recent research findings and advanced sediment transport methods. The method was subsequently modified and applied to the Dundee Central Area Sewer Model catchment as a test case.

The modifications made to this method may be summarised as follows:

- Ability to account for non-circular sections;
- Computation of boundary roughness for bed and walls, allowing for bed-forms;
- Utilisation of transport formulae for conditions other than at the limit of deposition;
- The ability to characterise (by size) the type of material likely to be deposited at any location;
- The updating of the critical erosion values used to reflect findings of cohesive bed research.

As the procedure relies on the output files from HydroWorks, a spreadsheet format was developed for the model. This was also necessary as HydroWorks does not allow the user a method of identifying the relative position of a pipe within the network unlike InfoWorks software, which allows an automatic upstream trace to be carried out.

The results of all previous sediment surveys carried out in the Dundee Area were re-analysed to determine an average pattern of sedimentation for the city centre. In addition to this, interviews of experienced water authority staff were carried out to determine the locations known to be persistent areas of sedimentation. This was done in an attempt to eliminate the normal "snapshot" picture of sediment deposition where seasonal or operational variations in sediment levels can be significant.

These final data were then used as a basis for the calibration of the model (Figure 4.2). The calibration was a difficult and laborious exercise as the manual mapping of shear stress and sediment deposit data for each pipe for a range of events was required at each stage of the analysis. This was found to be a significant drawback of the approach and is likely to limit its use by sewerage practitioners in its current form. The automation of this process would make implementation of this method (or similar) much more practical for large catchments. This would now be possible with the additional GIS capabilities of InfoWorks or the flexibility offered by some recent GIS applications (e.g. MAPBASIC).

Initial difficulties were experienced due to the large number of variables involved. As a result of this, as many variables as possible were kept constant. Items of highest perceived variability (i.e. particle diameter and specific gravity) were used as calibration parameters. This assessment of variability was based on observations of previous studies, accuracy of sampling and accuracy of measurement. However, due to the high number of potential combinations of the particle diameter and specific gravity variables, initial tests centred on the use of previously collected Dundee City Centre averages (detailed in Chapter 2) and the suggested default figures used by Gent and Orman (Gent & Orman, 1991). The Dundee City Centre sediment averages are summarised in Table 4.1, with the Gent and Orman default sediment characteristics given in Table 4.2.

Parameter	Perceived Variability	Data Source	Procedure Adopted
Particle Sizes	High	Dundee Studies	Variable - Initially use local data
Sediment SG	High	Dundee Studies	Variable - Initially use local data
Pipe Roughness	Low	Dundee Studies	Fixed - Use HydroWorks data
Bed Roughness	Medium	Dundee Studies & Experimental Work	Fixed - assumed k_b=50mm
Critical Shear	Medium	Dundee Studies & Experimental Work	Fixed - Use Gent & Orman figures

Table 4.1 - Calibration variables

Particle Size	d_{50} = 417 μm	
Sediment SG	SG = 1.58	
Pipe Roughness	HydroWorks values	
Bed Roughness	k_b = 50mm	
Critical Shear	Storm 90 = 2.5 N/m^2	Storm 15 = 9 N/m^2

Table 4.2 - Data Set 1 (Dundee measured averages)

The results using measured average figures proved to be disappointing, as sedimentation was not predicted in any of the pipes within the system. The dry weather erosion criterion was met in all but the most upstream of pipes. However, as all dry weather sediment sources are deemed to be in-pipe within the analysis, if a condition of erosion does not exist upstream, sedimentation cannot occur within the model. It should be noted that when considering the storm criteria, particle size and density play no part in determining the erosion of sediment (unless used to determine bed roughness).

As no meaningful results were achieved using the as measured Dundee data, it was decided that the 'blind' alteration of particle sizes and densities would prove unjustified and extremely time consuming. As an alternative to the Dundee data (sampled as large bulk samples), the default values used by Gent and Orman were tested. These figures were taken from the MOSQITO user's manual (HR Wallingford, 1991) and are summarised below in Table 4.3.

	Size (mm)	Density (kg/m^3)
Coarse	3.5	2650
Fine	0.5	1020

Table 4.3 - Wallingford Sediment Data

It has been reasoned that as a result of the selectivity of sediment washoff and transport, up to 97% of sewer sediments are coarse (Gent & Orman, 1991). Consequently, within the Gent and Orman procedure all sediments are assumed to be coarse. This gives rise to data set 2 (see Table 4.4).

Particle Sizes	D_{50} = 3.5 mm
Sediment SG	SG = 2.65
Pipe Roughness	HydroWorks Values
Bed Roughness	k_b = 50mm
Critical Shear	Storm 90 = 2.5 N/m^2
	Storm 15 = 9 N/m^2

Table 4.4 - Data Set 2 (default values)

The differences between the two sets of sediment data are very marked. Particle size has increased by a factor of nearly 8.4, and specific gravity by a factor of 1.7. This produces particles with a much lower propensity for erosion. In order to compare the propensity for erosion of the two data sets, it is necessary to consider the ratio of Ackers and Whites' total load transport coefficients, F_{gr} to A_{gr}. Further details of the coefficients proposed by Ackers and White are provided in Section 2.6.3.5. These coefficients are used to give a measure of the eroding and restoring forces (respectively) applied to a particular sediment grain. The higher this ratio for the same hydraulic condition, the more likely erosion is to occur.

	F_{gr}/A_{gr}
Data Set 1	1.517
Data Set 2	0.303

Table 4.5 - Comparison of Sediment Characteristics

Table 4.5 shows that the sediment characteristics of data set one produce particles approximately 5 times more likely to erode than those of data set two. Although the mean particle size and density used in data set two are significantly greater than those measured in the field (in Dundee), it is necessary to consider the overall effect of this.

The criteria used to determine the onset of erosion in the Gent and Orman method (Ackers and White) were initially developed for use in rivers, and hence do not explicitly account for the effects of cohesion, agglutination and cementing usually found in sewer sediments. By increasing the size and density of particles used in the analysis, these effects are likely to be indirectly accounted for. These effects are likely to play a role even in daily erosion as cohesive strength has been observed to re-establish in a matter of hours rather than days (Wotherspoon & Ashley, 1992; Williams & Williams, 1989; Stotz & Krauth, 1986).

It is also proposed that the use of such large particles removes the sensitivity of the procedure to the dry weather phase. As such large dense particles are used, widespread dry weather deposition was often found to be predicted. This list of locations is then refined through the storm analysis. However, the field investigations of this study have indicated that this dry weather stage of the analysis overpredicts depositional patterns. This becomes less important as the storm stages of the method progress, as these stages have been found to dominate the analysis.

4.4.1 Model Results

Figure 4.3 (below) shows predicted areas of likely deposition highlighted in red. These results are for the system without considering the effects of tides, and therefore the sewers in the lower parts of the system will not be represented correctly. However, this was done as future engineering enhancements to the system were planned to alleviate tidal ingress.

Figure 4.3 may be compared with Figure 4.2 in order to evaluate the performance of the procedure. On immediate inspection, it is clear that the geographical locations of the areas of deposits are similar in both cases. Although deposition is correctly predicted in the Murraygate area of the interceptor sewer (pipe refs. 401_080.1 to 401_160.1), there are slight discrepancies regarding the actual pipes predicted (e.g. predicted deposition in pipes 401_150.1 and 401_160.1 and not in pipes 401_010.1 to 401_140.1).

The largest differences are apparent when examining the Dock Street sewer (leg 601). Although deposition is predicted in this area, sediments do not appear on the 1989 survey summary. However, experience has shown that deposits do exist in this area and that the lack of survey data is due to surveying difficulties created by the tidal regime operating in these pipes. "Permanent" pipe deposits in this area have been recorded by operational staff carrying out repair work to the Dock Street sewer. Surveyed sediments located in the Seagate area are represented in the procedure by the deposition predicted in pipe 611_030.1.

Surveyed	Sediment Location	Predicted
Y	Murraygate Interceptor	Y
Y	Seagate	Y
Y	Blackscroft	Y
Y	Union St./ Whitehall St.	
Y	Perth Road (upper)	Y
Y	Perth Road (lower)	Y
Y	Polepark	Y
Y	Dens Road	
Y	Dura Street	
Y	Dock Street	Y

Table 4.6 - Comparison of Surveyed and Predicted Deposition from modified Gent and Orman method

As can be seen from Figure 4.2 and Figure 4.3, the majority of the areas of sedimentation have been predicted by the procedure. Table 4.6 demonstrates that the largest area not included in the predicted areas of sediment is that of the Dens Road area. Most of the pipe lengths subjected to deposits in this area were discounted in the dry weather stage of the procedure. The steep pipe gradients of the pipes in this area do not logically lend themselves to sediment problems. However in this particular case it is the relative gradients and local flow features that are of most relevance. Consequently it is believed that the deposits in this location are storm deposits only. In addition to this the principal location missed in this case is located just upstream from a complex junction including a 90-degree bend. Features such as this, although modelled hydraulically, were not fully represented by the analysis. This is believed to result from hydraulic modelling limitations (incorrect model headloss coefficients and un-modelled watercourse interaction) and the selection of only one characteristic particle size.

Figure 4.2 - Measured sediment locations in Dundee City Centre

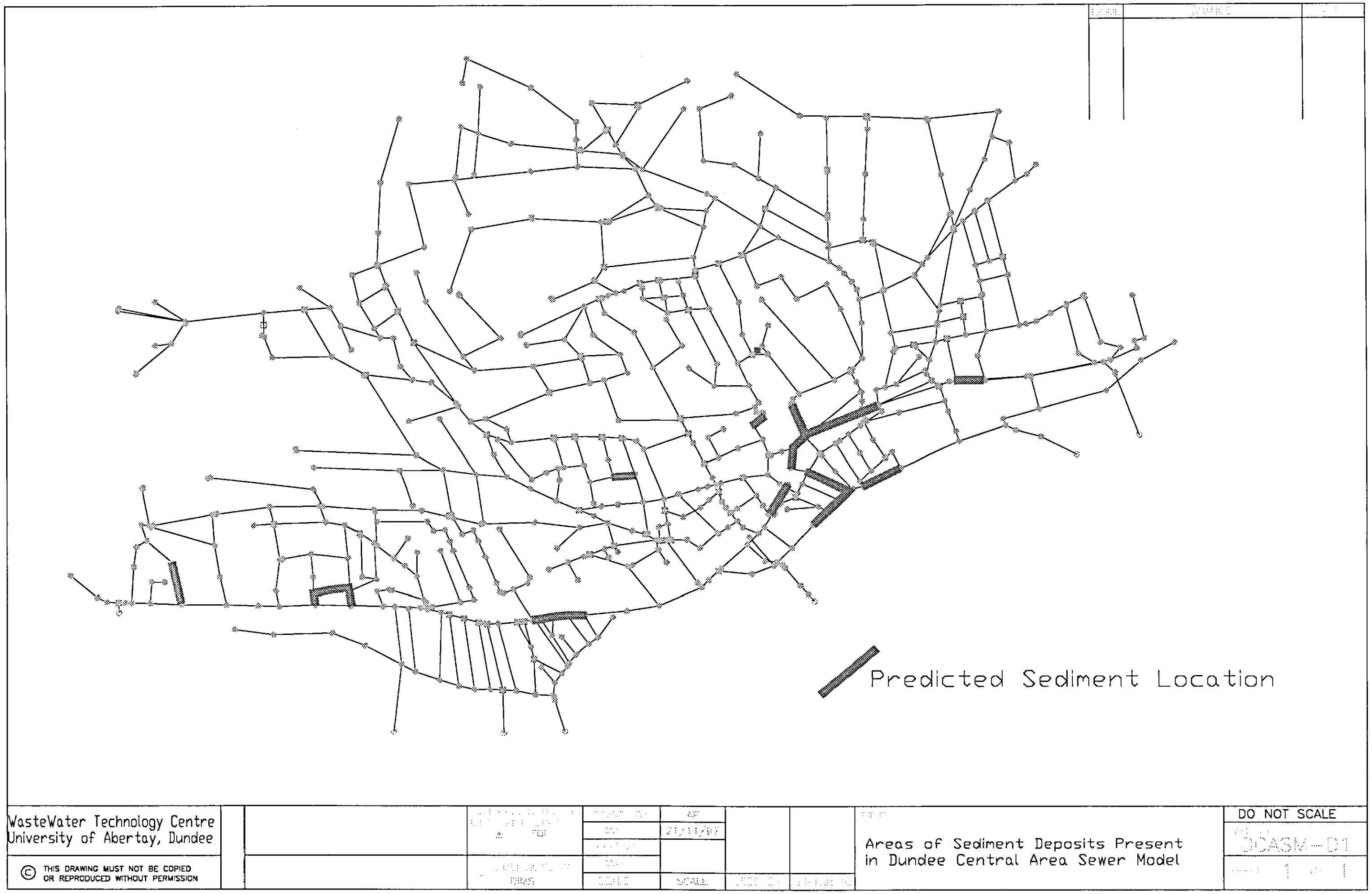
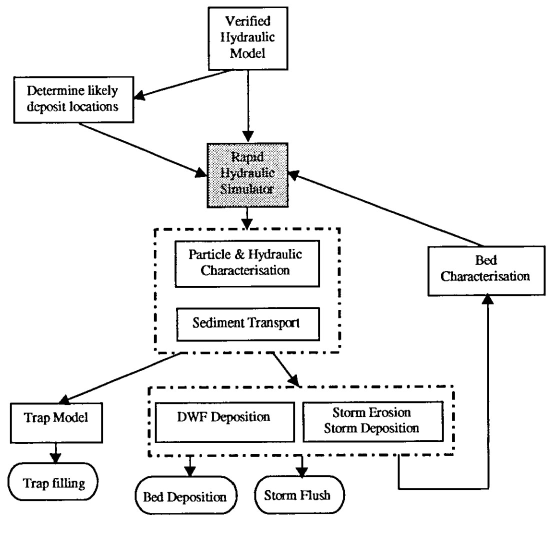


Figure 4.3 - Predicted Sediment Locations (Modified Gent & Orman)

4.5 *Rapid Hydraulic Simulator*



The modelling of the evolution of sediment deposits requires simulations of durations in excess of 1 year. This results from the time taken for some deposits to develop to problematic levels and the likely infrequent maintenance strategies that require to be tested. Some hydraulic models are limited by the duration that can be continuously simulated. In addition to this, long-term detailed simulation of large drainage systems can result in excessive simulations times. This can be particularly impractical when scenario modelling is being undertaken. To facilitate more efficient long-term simulations times, a rapid hydraulic simulator was developed as a hydraulic driver for the sediment models.

The key requirements of the hydraulic driver were:

- Detailed hydraulic outputs (flow, depth and velocity) at 2 minute intervals in order to allow the accurate representation of storm conditions;
- Continuous simulation to represent dry weather and storm flows;
- Long-term simulation durations, as sediment beds may take months or years to evolve;
- Short simulation run times are required as detailed models are currently impractical to use for scenario modelling;
- Continuity of data transfer between hydraulic and sediment model;
- Ability to feed sediment modelling results back into hydraulic model.

Various options that could be used as hydraulic drivers for the sediment models were assessed. Initially it was expected that the HydroWorks modelling suite could be adapted to allow the most commonly used hydraulic modelling methods to be built into the approach. However, at the time of model development, HydroWorks was incapable of providing simulations for any period greater than 28 days. This made the use of HydroWorks impossible. Since then, InfoWorks has been developed to allow longer simulation durations. However the processing time required for these models to run does not allow for practical modelling when considering durations of the order of one year.

It was therefore decided to persevere with HydroWorks when carrying out the pipe by pipe analysis required to identify potential sedimentation locations, but to use a simplified hydraulic model to provide the detailed, long-term analysis.

A range of simplified modelling tools was examined for suitability. Discussions with project collaborators in Liverpool highlighted the potential use of a simplified hydraulic model developed using unit hydrograph theory (Mehmood, 1995).

4.5.1 Unit Hydrograph Theory

In simple terms, the unit hydrograph method allows a conversion to take place between a depth of rainfall and the runoff from the catchment surface that results. This is done through calculating the quantity of runoff from a "unit" of rainfall (often 1 mm) and superimposing these runoff hydrographs to give the total runoff for a catchment area.

The approach was first introduced by Sherman in 1932 (Sherman, 1932). Traditionally the runoff element of the approach was estimated using records of rainfall and recorded runoff for large catchment areas (Shaw, 1994). However in this work, a verified HydroWorks model was used to determine the runoff volumes to allow the procedure to be calibrated. The HydroWorks runoff models represent relatively detailed processes such as permeability, antecedent conditions, changing wetness during rainfall and surface gradient.

Within this process, a full HydroWorks model of a drainage system must be simplified into a number of manageable discrete subcatchments. The level of simplification will depend upon the intended use of the model. In its simplest form, the model could produce the runoff volumes at only one point of interest (e.g. a deposition location).

To derive a unit hydrograph that includes details of runoff, storage and routing, a suitable duration of event should be selected. It is necessary that the entire catchment area contributes to the unit hydrograph; therefore the time of concentration is typically used as the event duration.

As the hydrograph must accurately generate runoff over a range of rainfall intensities, an intensity equal to that of the maximum recorded value (or maximum intended design intensity) should be used (Mehmood, 1995) to derive the unit hydrograph.

In order to eliminate the effect of initial losses, the unit hydrograph is derived as the difference between the runoff resulting from two rainfall events. The first including the desired unit rainfall preceded by a nominal depth of rainfall to satisfy the initial losses, and the second, simulating only the initial nominal rainfall flows.

Once the unit hydrograph is created, it can be used to generate flow for recorded or design rainfall events. At each timestep, the unit hydrograph is convoluted with the rainfall profile used in the event to produce runoff. A sample of how these calculations are used is shown in Figure 4.4.

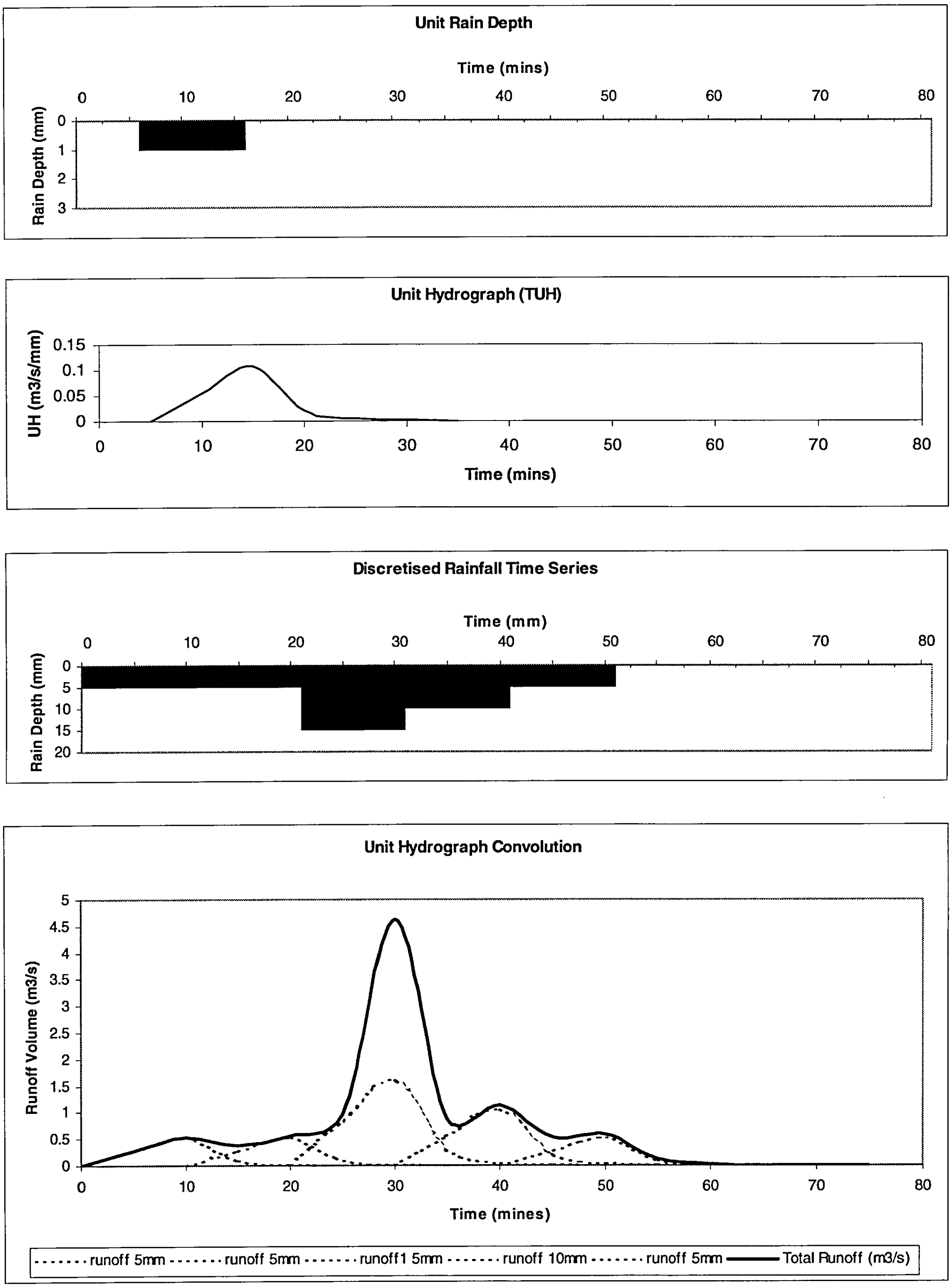


Figure 4.4 - Runoff synthesis using the unit hydrograph approach

4.5.2 Model Calibration and Testing

The unit hydrograph approach is characterised by a number of simplifications such as the absence of details of catchment wetness, land type and slope. However, as using the approach suggested here involves calibrating the unit hydrograph model against a detailed hydraulic model that does contain these elements, these details are indirectly represented.

The variables used within the approach for the rainfall to runoff conversion are provided below:

Variable	Source / Comment
Time of concentration (Tc)	Derived using verified hydraulic model
Tc unit hydrograph	Derived using verified hydraulic model
Rainfall	Recorded or synthetic continuous input
DWF inputs	Derived using verified hydraulic model

Table 4.7 - Summary of hydraulic model input variables

The routing of runoff volumes is represented simply via a conservation of volume approach, with an appropriate time delay chosen in each link element to represent the required level of system storage. The delay times in this study were calculated by determining the transfer times for peak dry weather flows using the verified hydraulic model. The various other components of the simplified catchment model are described further later in this section.

As previously discussed, the sewerage model of Dundee's central area was used for model testing and development. The network has has a number of characteristics, which were anticipated to cause problems for the unit hydrograph modelling procedure, including:

- Looped flow routes;
- Numerous flow control gates;

- Flow splits created by bifurcations;
- Tidal influence causing backwater effects.

In an attempt to make the unit hydrograph method more flexible, the techniques used to describe how the flow was routed through the system were modified to allow for the modelling of bifurcations and half gates. As the initial model was coded within the MATLAB programming environment (Mehmood, 1995), this platform was retained and used for the development of the other component models to ensure continuity and ease of data transfer. The environment was however adapted slightly to allow graphical programming to be used. This was deemed suitable as it allows complex drainage systems to be built up within the model more easily as a geographical view of the system can be used rather than simple text. The graphical programming environment used is known as SIMULINK and is part of the MATLAB modelling suite.

The full HydroWorks DCASM model was used as a template for the development of the simplified model, with catchments defined according to the discrete sub-models within the DCASM model. It was also used as a benchmark against which to calibrate the model for a range of storm events.

The first stage of the procedure was to break down the catchment into discrete subcatchment areas at a level of detail appropriate to the study. This level of detail will depend on the complexity of the catchment, the number of CSO's and the number of locations where detailed hydraulic simulation data are required. As Dundee was being used as the initial test, all major subcatchment areas were represented, in order to provide a significant test for the software and to determine if detailed hydraulic results could be calculated for the main interceptor at the base of the catchment.

Model build reports for the Dundee Central Area Sewer Model (DCASM) were reviewed to allow the most appropriate subcatchments to be identified. This followed the pattern set by the major flow junctions and flow survey positions in order to allow access to initial flow survey data if required. Figure 4.5 (below) shows the initial definition of subcatchments in the central Dundee area as determined by the sewerage operator (Scottish Water). Figure 4.6 shows a schematic of how these subcatchments are represented in a simplified model.

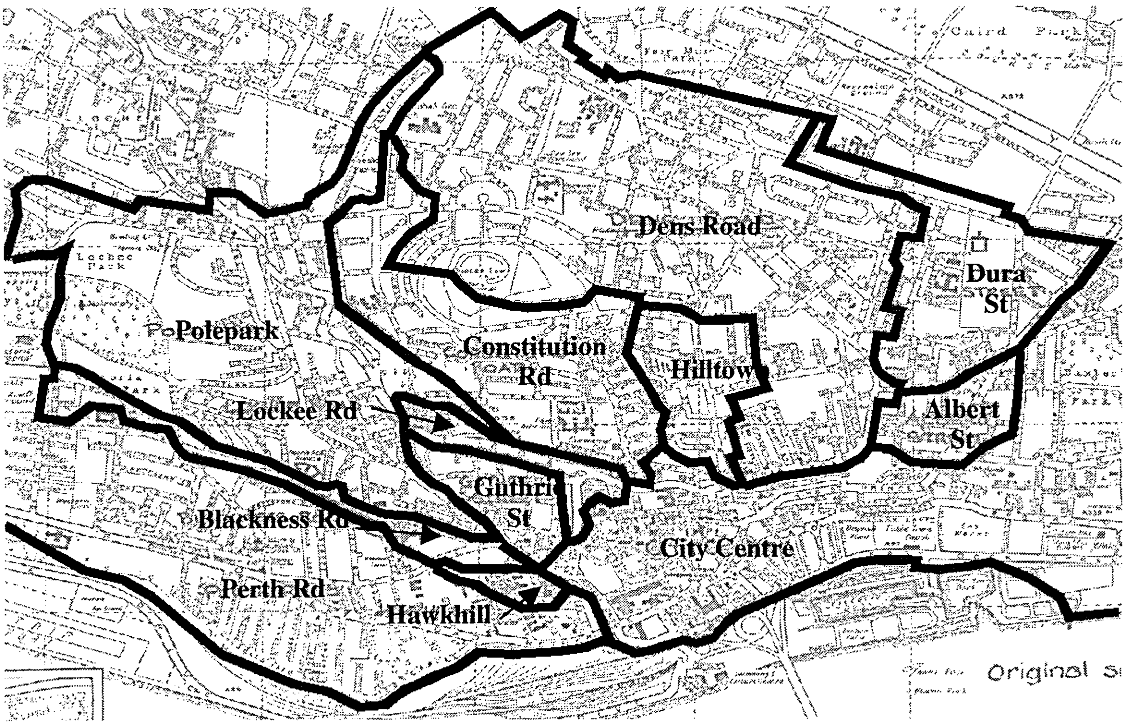


Figure 4.5 - Definition of DCASM subcatchment boundaries

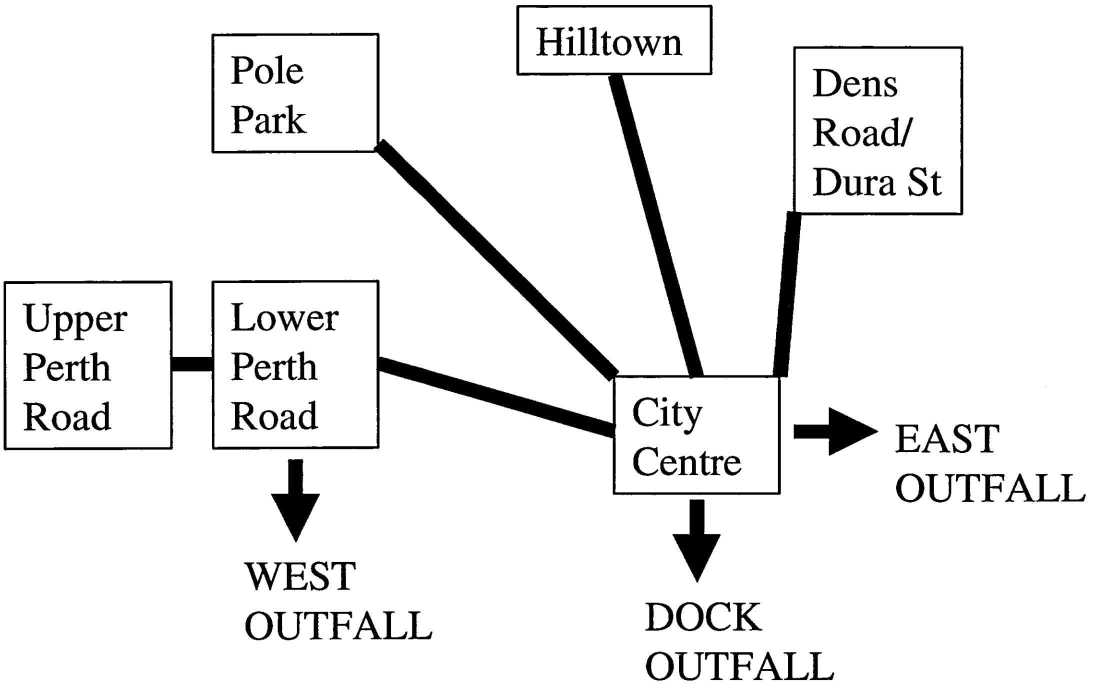


Figure 4.6 - Simplification of DCASM Subcatchments

The network was defined using the following principal modelling elements:

- Catchment area inputs (converting rainfall to runoff);
- Junctions (allowing the adding of joining flows);
- Pipe delays (to allow travel time and storage between junctions to be represented);
- Overflows (also modified and used to represent half gates);
- Bifurcations.

The resulting catchment model of Dundee's City Centre was termed "FASTDCASM" and is shown diagrammatically in Figure 4.7. The diagram is the uppermost layer of the model and shows the various catchments and components combining together to represent the DCASM catchment. The sediment module can also be seen extracting flow data at a deposit location. This module is discussed further in the following sections of this chapter.

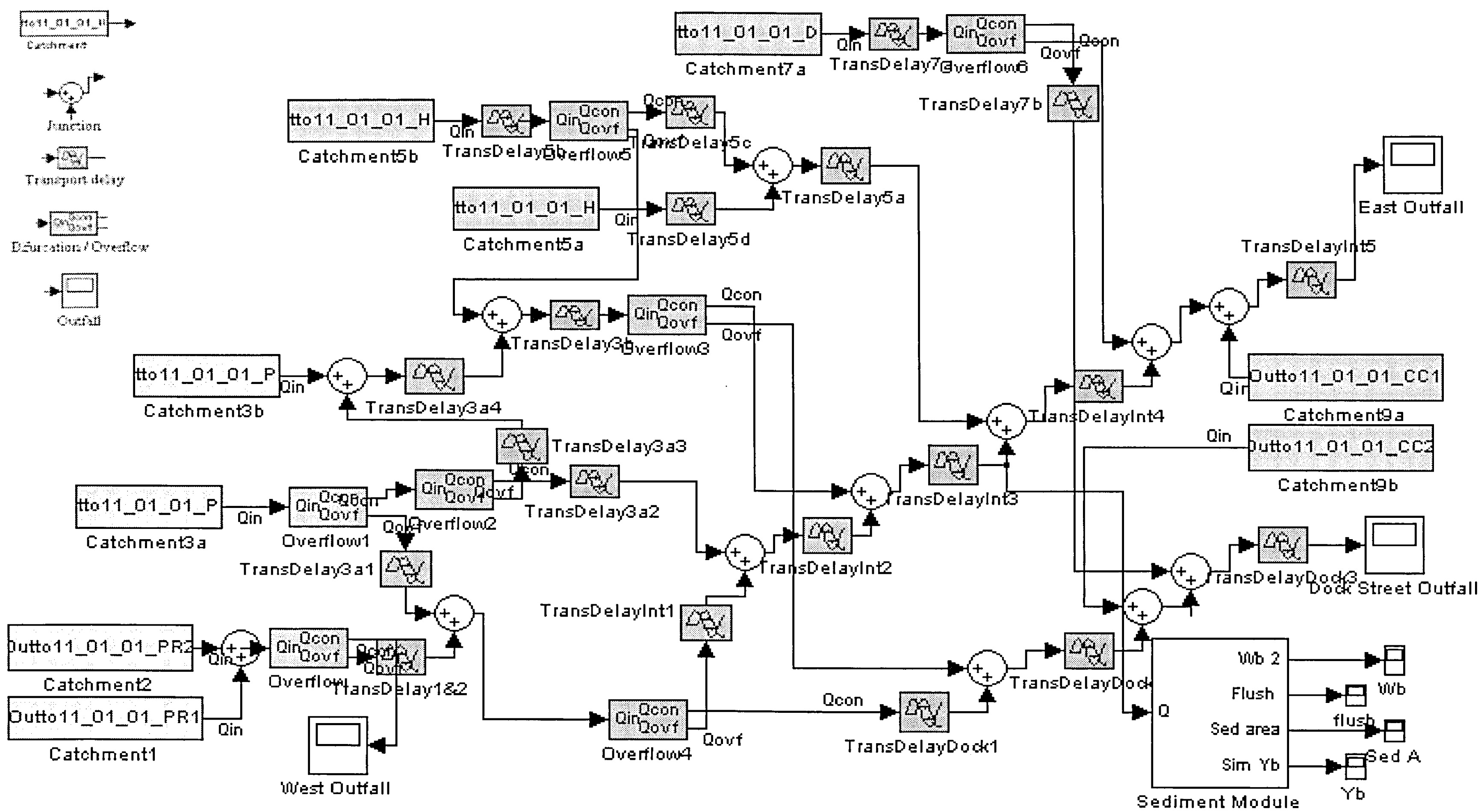


Figure 4.7 - Representation of Dundee Central Area Sewer Model in FASTDCASM

4.5.2.1 Model Calibration

The calibration process for the FASTDCASM model is as follows:

- Define appropriate subcatchment boundaries and additional points requiring detailed calculations;
- Construct a skeletal model using the model components described above;
- For each subcatchment area calibrate outflows against HydroWorks model outputs using the variables of the time of concentration (Tc) and the resulting Tc unit hydrograph.
- Calibrate flows within the network at key locations (e.g. overflows and deposit locations) using the variables of the CSO's, bifurcations and the time of travel in conduits.

Within this procedure, the runoff from each catchment is pre-processed in MATLAB prior to being routed using the network model coded in SIMULINK. A continuous series of rainfall data at 2-minute intervals (including zeros) is used to produce the time / runoff matrix for each subcatchment in turn. These data are then saved and used in the routing model. This has the advantage in the case of 1 year of continuous design rainfall in that it need only be processed once, resulting in fast simulation times.

The simplified model was calibrated against the HydroWorks model using five design events of varying return periods and flow only as an initial calibration parameter. These events were chosen to represent approximately weekly, monthly, quarterly, six monthly and annual rainfall events. The system outfalls were used for model calibration in order to allow all components to be included. The operational version of the model used at that time allowed nearly all flow to pass to the Dock Street Outfall and East Outfall.

The outputs from the unit hydrograph model were calibrated against the outputs of the HydroWorks model for each of the 9 subcatchments for each calibration event. The principal variable used for calibration at the subcatchment level was the time of concentration. In order to determine an appropriate range of values for the time of concentration, a series of sensitivity tests were carries out using a range of rainfall events simulated in the HydroWorks model. In general, the sensitivity of the model to changes in the time of concentration was found to decrease with peak rainfall intensity. Consequently, it is recommended that the time of concentration associated with the largest rainfall event in the series to be simulated is used initially to derive the Tc unit hydrograph profiles. The time of concentration was then adjusted to provide an improved fit for hydrographs at each subcatchment.

Following the initial calibration of the outputs from each subcatchment, the performance of the bifurcations and transport delay times were altered to provide a level of flow routing and attenuation comparable with the HydroWorks model. This was an iterative process where the mean pipe velocities and distances between major junctions were determined in HydroWorks. This allowed transport delay times to be calculated and applied to the simplified model.

The bifurcations within the simplified model were adjusted to better represent the split of flows observed in the HydroWorks model. As the model element used to represent the bifurcations was essentially a modified CSO structure, no attempt was made to ensure that the parameters used to achieve the split were accurate (e.g. spill pipe level and pipe diameter). The variables used within the CSO/bifurcation model are given below:

Variable	Source / Comment
Chamber area	Sewer records or model data
Weir / overflow height	Sewer records or model data
Continuation pipe area	Sewer records or model data
Continuation pipe Cd	Model data or engineering estimate
Continuation pipe free flow capacity	Model data or engineering estimate

Table 4.8 - CSO/bifurcation model input variables

4.5.2.2 Model Calibration Results - DCASM

Table 4.9 and Table 4.10 (below) show the total volume results for the flow verification for the Dock Street outfall and East outfall respectively. In order to assess the errors associated solely with the simplified modelling approach, the results of the HydroWorks model were used as a baseline. In reality, the errors from real observed flows will vary from those shown in the following tables as a result of further inaccuracies of flow measurement and full solution modelling in HydroWorks.

The general performance of the model in both cases appears to be an underestimate of total flow volume for the larger events and an overestimate of volume for the smaller events. This may be as a consequence of the selection of the time of concentration for each subcatchment (see Section 4.5.2) or as a result of the variability of catchment wetness throughout storms becoming more significant for the larger events.

Although large events can have significant effects on the movement of sediment, their infrequency did not warrant heavy weighting of the calibration of flow volumes toward these events.

Current modelling guidelines for full solution modelling suggest a total volume accuracy of between +20% to –10% (WaPUG Code of Practice, 2002). Although these results generally fall outwith these parameters for the larger events, given the simplified nature of the modelling and the complexity of the system modelled, the results for Dundee are considered generally acceptable. As can be seen, a higher degree of accuracy was achieved at the East Outfall. It is believed that this is a result of the Dock Street outfall accepting more "spill" flows from bifurcations than the East Outfall. Consequently, the simplified method of bifurcation representation may have a greater effect.

Event Return Period	Total DCASM Volume (m^3)	Total FASTDCASM Volume (m^3)	Difference (m^3)	% Difference
Annual	13265.5	10521.7	-2743.9	-21
6 Monthly	11299.3	9116.5	-2182.7	-19
Quarterly	8209.7	6659.7	-1549.9	-19
Monthly	3500.3	2926.9	-573.3	-16
Weekly	1024.4	1212.9	188.6	18

Table 4.9 - Dock Street outfall FASTDCASM total volume results

Event	Total DCASM Volume (m^3)	Total FASTDCASM Volume (m^3)	Difference (m^3)	% Difference
Annual	8495.8	7714.4	-781.4	-9
6 Monthly	7725.2	7143.9	-581.3	-8
Quarterly	6431.9	6114.5	-317.4	-5
Monthly	4202.4	4270.5	68.1	2
Weekly	1397.0	1781.9	384.9	28

Table 4.10 - East outfall FASTDCASM total volume results

As the erosion model chosen is concerned principally with the peak conditions for any particular event, accuracy of peak conditions is also of significant importance. Examination of Table 4.11 and Table 4.12 reveals a reasonable level of accuracy generally within the range of +25% to –15% suggested in the WaPUG code of modelling practice. The significant exception to this is the peak flows produced for the weekly event at the East outfall. Although a sensitivity analysis has shown the smallest events to be the most sensitive to calibration changes, an examination of the performance of the model at the input locations of the unit hydrographs reveals errors in the order of +10% at each site.

A further calibration factor was examined to explain this poor comparison of flows at the catchment outlet. The transport time in each of the modelled links were used to represent the level of storage in the system. Average travel times were calculated over the range of events used. It is therefore suggested that for the smaller events, these transport times were underestimated resulting in exaggerated peaks. But again as a balance of results over the year was sought and overestimation was not observed in other parts of the catchment (i.e. deposition locations), only minor changes to the travel times were made to reduce this effect for the one event.

Figure 4.8 to Figure 4.26 show the plots of all calibration events. It should be noted that the approximate shapes of all hydrographs are replicated indicating the correct proportions and timings of the various catchments. However a shift in time is apparent in the majority of the figures indicating that the flow is routed too slowly. This is supported by the fact that the lag times are most pronounced during the largest events which would naturally give rise to the fastest velocities. Adjustments made to the travel times resulted in reduced lag but detrimentally affected the profile shapes. As the simulated durations are in the order of 1 year it was decided that this lag time would have little or no effect on the overall results.

The only poor level of fit was observed at the East Outfall when using the weekly event. Under these conditions, the MATLAB model was found to overestimate the peak flow by approximately 50%. However, the improvement of this weekly simulation was found to significantly underestimate not only flows for the other simulations, but also other locations within the weekly simulation.

Return Period of Event	Peak DCASM Flow (m^3/s)	Peak FASTDCASM Flow (m^3/s)	Difference (m^3/s)	% Difference
Annual	4.307	5.3543	1.0473	24.3
6 Monthly	4.024	4.1793	0.1553	3.9
Quarterly	3.25	3.4784	0.2284	7.0
Monthly	1.603	1.2652	-0.3378	-21.1
Weekly	0.396	0.4891	0.0931	23.5

Table 4.11 – Dock Street outfall FASTDCASM peak flow results

Return Period of Event	Peak DCASM Flow (m^3/s)	Peak FASTDCASM Flow (m^3/s)	Difference (m^3/s)	% Difference
Annual	3.051	3.2937	0.2427	7.9
6 Monthly	2.705	2.5028	-0.2022	-7.5
Quarterly	2.158	2.169	0.011	0.5
Monthly	1.312	1.6427	0.3307	25.2
Weekly	0.352	0.5291	0.1771	50.3

Table 4.12 - East outfall FASTDCASM peak flow results

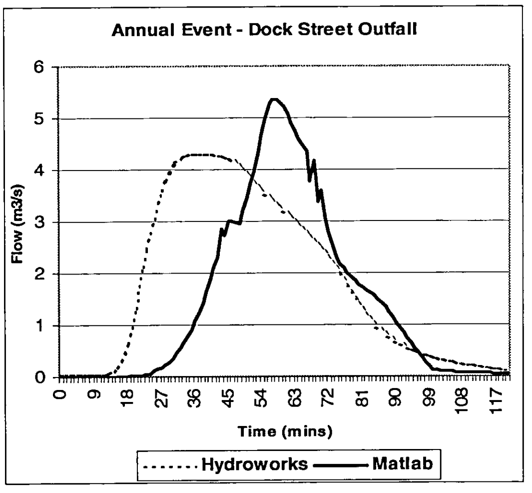


Figure 4.8 Dock St calibration – annual

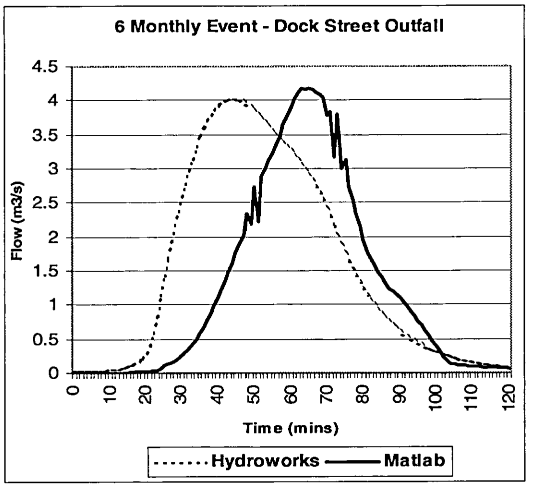


Figure 4.10 Dock St calibration - 6 monthly

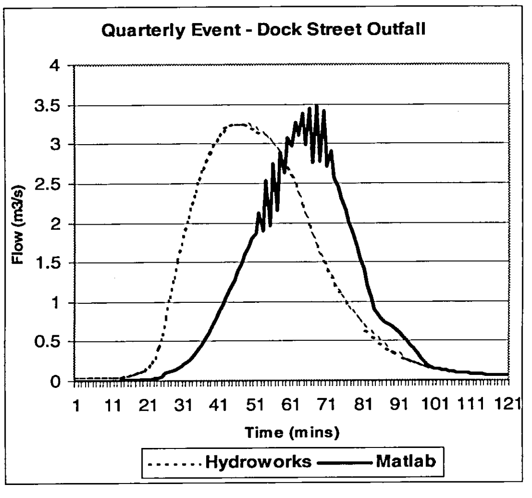


Figure 4.12 Dock St calibration – quarterly

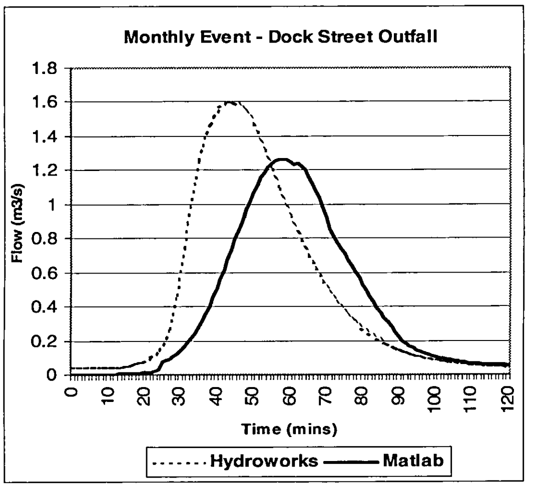


Figure 4.14 Dock St calibration – monthly

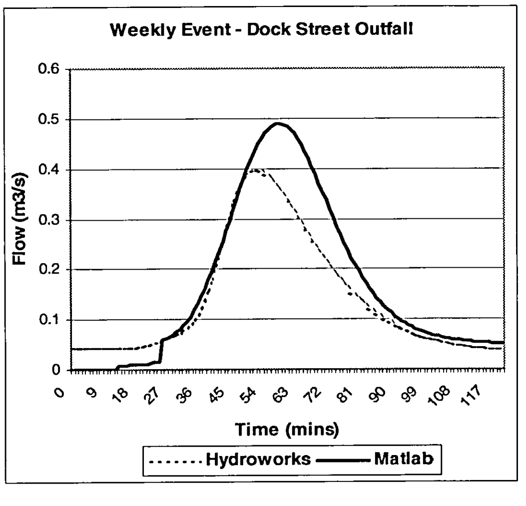


Figure 4.16 Dock St calibration - weekly

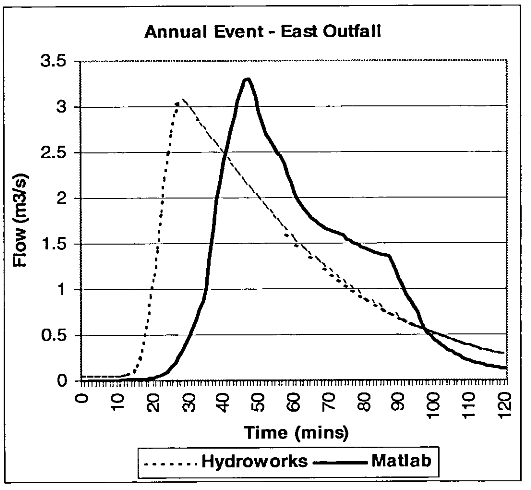


Figure 4.18 East calibration – annual

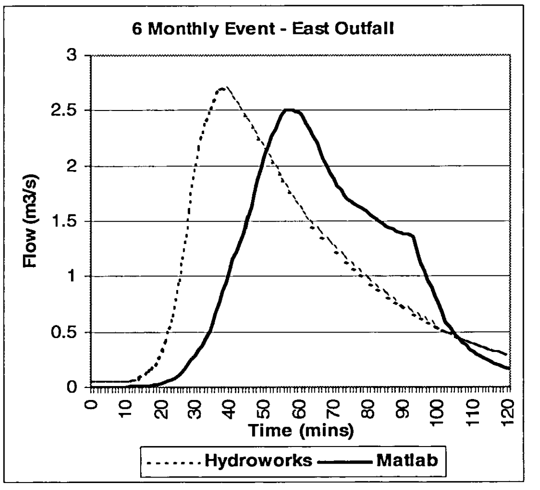


Figure 4.20 East calibration - 6 monthly

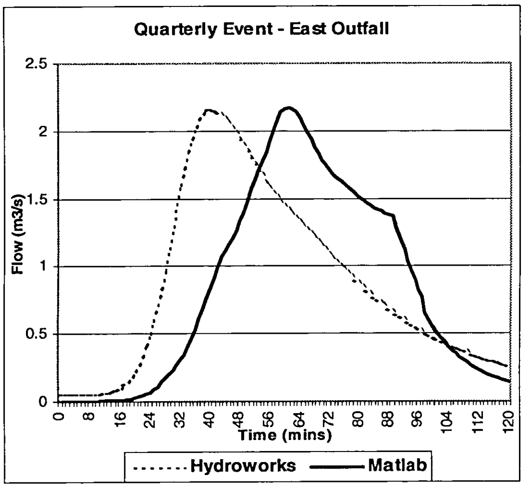


Figure 4.22 East calibration – quarterly

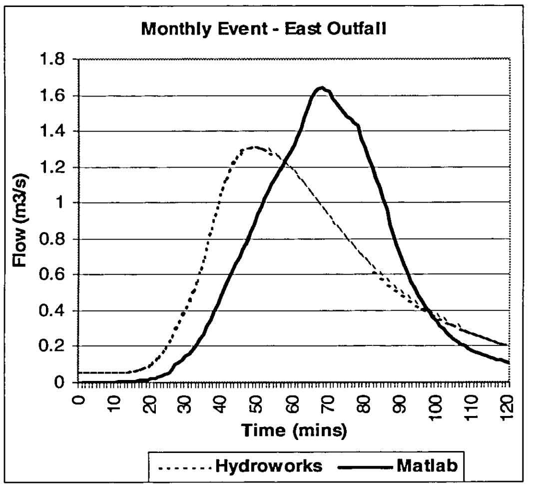


Figure 4.24 East calibration – monthly

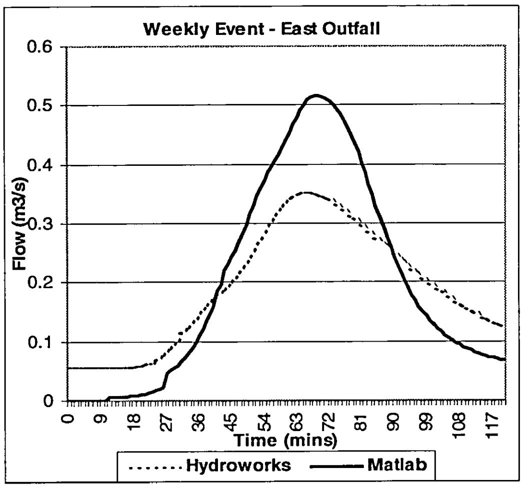


Figure 4.26 East calibration - weekly

4.5.2.3 Model Calibration Results – Forfar

As a further test for the approach, a simplified catchment model was also developed for the Forfar catchment. This was carried out as sediment studies were also being carried out within Forfar, and the Forfar drainage scheme offered a range of different tests for the procedure including:

- Mixed section shapes <900mm max dimension;
- Contains combined, separate and partially separate sub-systems;
- 10 main sub-catchments;
- Very interactive sub-catchments;
- 2 pumping stations.

Figure 4.28 shows a schematic for the Forfar Drainage system. Although much smaller in terms of population, area and number of pipes than the Dundee central area, the complexity of the Forfar system required a greater number of subcatchments and the development of pump model components.

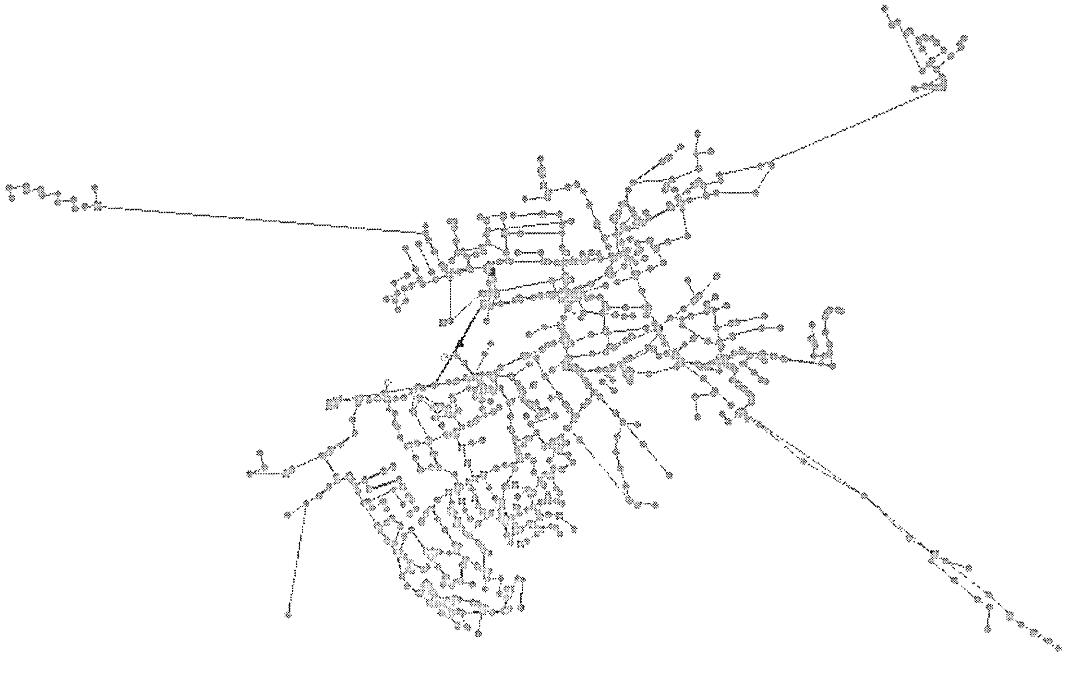


Figure 4.28 - Forfar HydroWorks Model

The same procedure that was used for developing the DCASM model was followed for Forfar, resulting in the FASTFORFAR SIMULINK model shown in Figure 4.30.

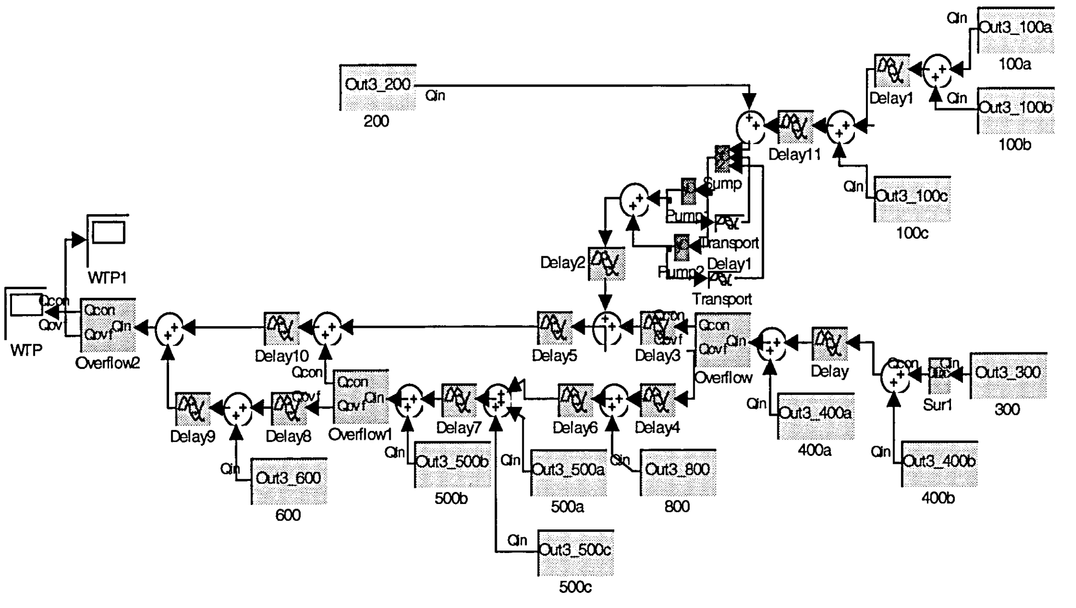
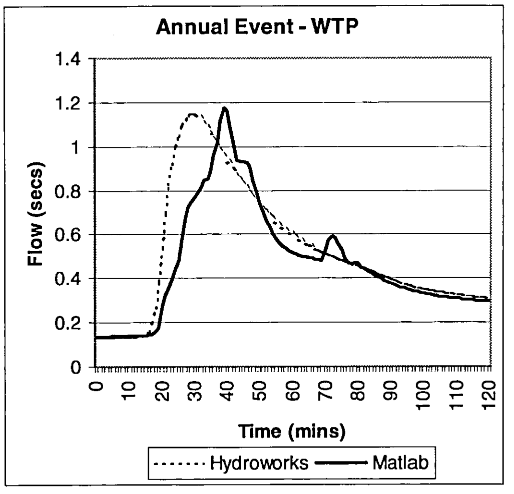


Figure 4.30 - Forfar SIMULINK model

Figure 4.32 to Figure 4.36 show a comparison of the HydroWorks and unit hydrograph modelled output for the same range of storm events used previously. A marginally better fit is experienced for the Forfar model, with the only exception occurring at the lowest intensity event.

Return Period of Event	Peak HydroWorks Flow (m^3/s)	Peak FASTFORFAR Flow (m^3/s)	Difference (m^3/s)	% Difference
Annual	1.184	1.195	0.011	0.9
6 Monthly	1.073	0.934	-0.139	-13.0
Quarterly	0.861	0.807	-0.054	-6.3
Monthly	0.618	0.578	-0.04	-6.5
Weekly	0.313	0.389	0.076	24.3

Table 4.13 – Forfar WTP FASTFORFAR peak flow results



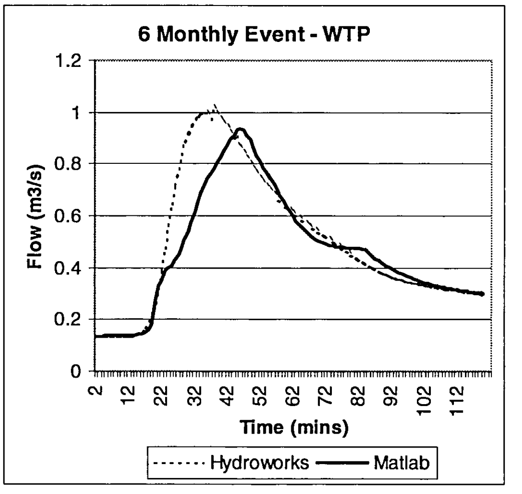
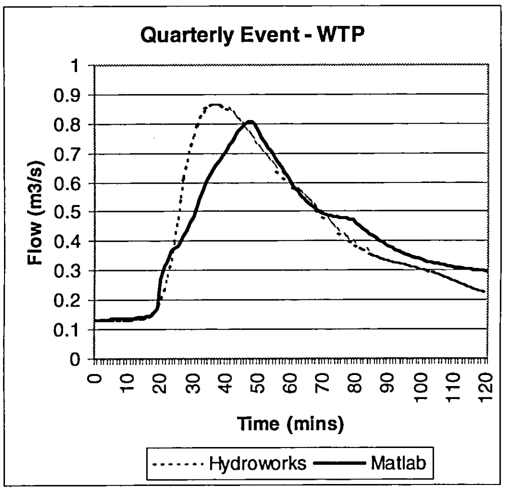


Figure 4.32 Forfar WTP – annual

Figure 4.33 Forfar WTP – 6 monthly



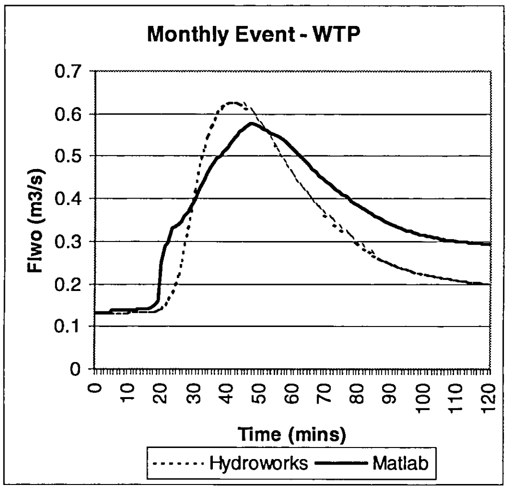


Figure 4.34 Forfar WTP – quarterly

Figure 4.35 Forfar WTP - monthly

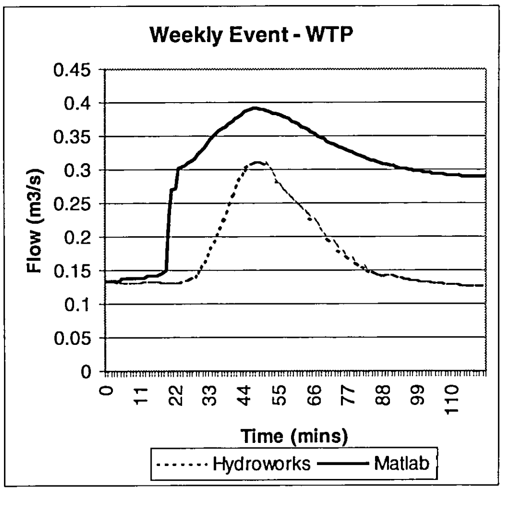


Figure 4.36 Forfar WTP - weekly

The performance of the simplified model has been tested against two fully verified HydroWorks models. The results presented here indicate that the level of performance is sufficiently close in terms of volume of runoff produced, to allow further calculations to be made using the unit hydrograph approach. However, in general it was noted that flows at the outlets of each model were overestimated for the smallest events. Given the good fits achieved for the other events and the detrimental effects on the other simulations of calibrating for this event, the principal cause for this behaviour could not be ascertained. However the effect is believed to relate to three main influencing factors:

- Increased influence of rainfall losses in the smallest events being underestimated;
- Unsuitability of the peaked unit hydrograph profile to produce the flat weekly event profile under convolution;
- Misrepresentation of bifurcations under lower flows.

Minor overestimation of flows (10-15%) was observed at a subcatchment level for the weekly event, with this cumulative effect noted at the base of the catchment. As the effect was reduced within the network at the locations where the detailed hydraulic data were required in order to determine sediment deposition rates, the method was developed further.

4.5.2.4 Deriving Flow Depths and Velocities

As the simplified unit hydrograph model provides only volumes of flow at each timestep, flow depth and velocity must be calculated. The method used here uses Manning's coefficient "n".

For each pipe cross section where detailed calculations were required, a look-up table was created of the section conveyance factor ($AR^{2/3}$) versus flow depth, y. At each timestep the conveyance factor was calculated as:

$$AR^{\frac{2}{3}} = \frac{Qn}{\sqrt{i}} \qquad \textbf{Equation 4-1}$$

Where: A = cross sectional area of flow (m^2)

R	=	hydraulic radius (m)
Q	=	flow (m^3/s)
n	=	Manning's coefficient
i	=	bed gradient

The flow depth and area are then derived from the previously calculated conveyance factor, allowing the velocity to be calculated from the continuity equation. The hydraulics are then adjusted to account for the presence of sediment with flow area, wetted perimeter and roughness all recalculated.

Although approximate, this approach was applied using realistic Manning's coefficients for Victorian brick sewers (~0.017 to 0.025) and provided accurate results when compared to the HydroWorks model.

However, for final verification, the modelled data were compared against measured data, recorded over dry weather and storm conditions with minor calibrations made using the Manning number.

4.5.2.5 Model Verification – Murraygate Interceptor Sewer

Detailed hydraulic monitoring was undertaken in the Murraygate Interceptor sewer in Dundee as this site was later to be used for the monitoring of sediment deposition. The calibrated model was run using 2-minute rainfall data collected from a 0.2 mm tipping bucket rain gauge located in the catchment. A significant series of storm events occurred between 29/8/00 and 4/9/00. This period allowed both dry weather and storm conditions to be tested.

An initial comparison between the measured data, HydroWorks simulation and FASTDCASM simulation results revealed that there was less storm flow passing through the interceptor sewer than predicted by the HydroWorks model. Consultation with the Water Authority revealed the removal of a gate at the junction of Polepark and Lochee Road. This results in an uncontrolled flow split with approximately 65% of the flow now passing to the Dock Street outfall. The inclusion of this flow split enhanced the FASTDCASM modelling results significantly.

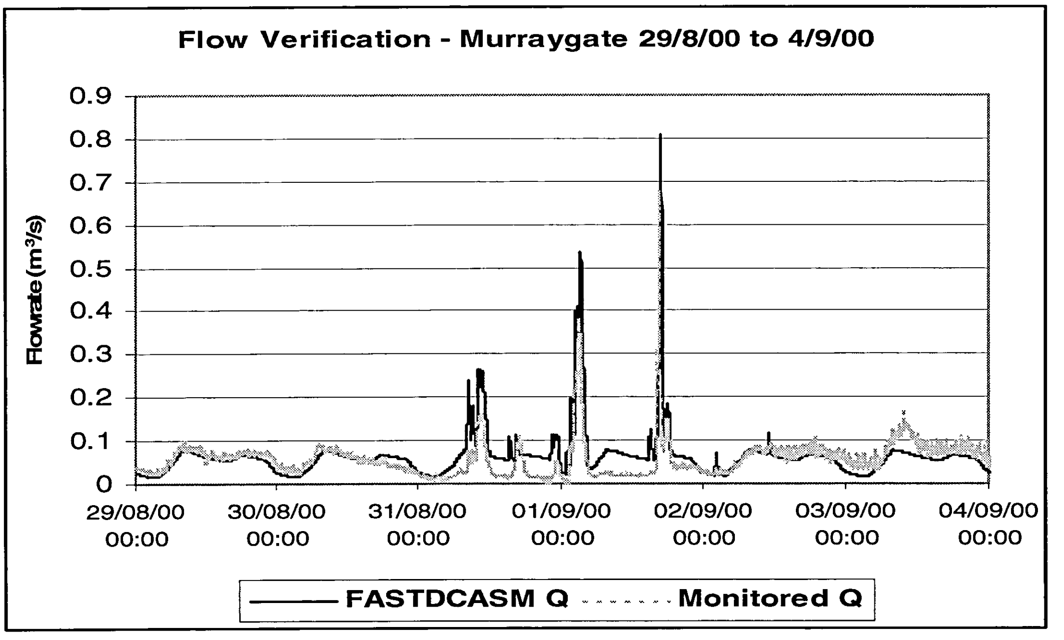


Figure 4.38 - Murraygate flow verification

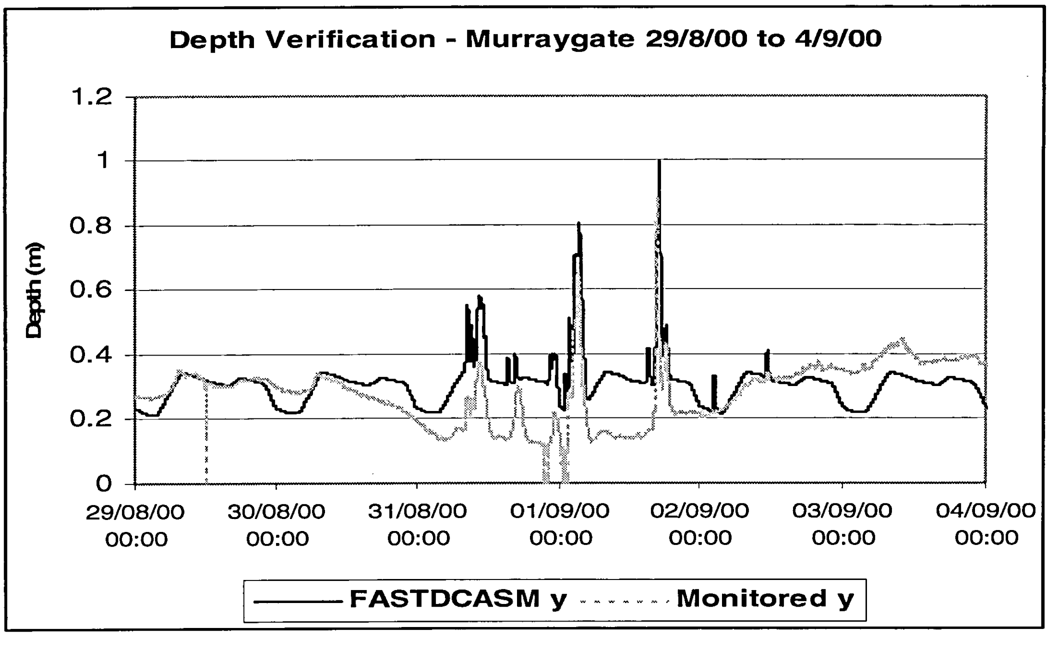


Figure 4.40 - Murraygate depth verification

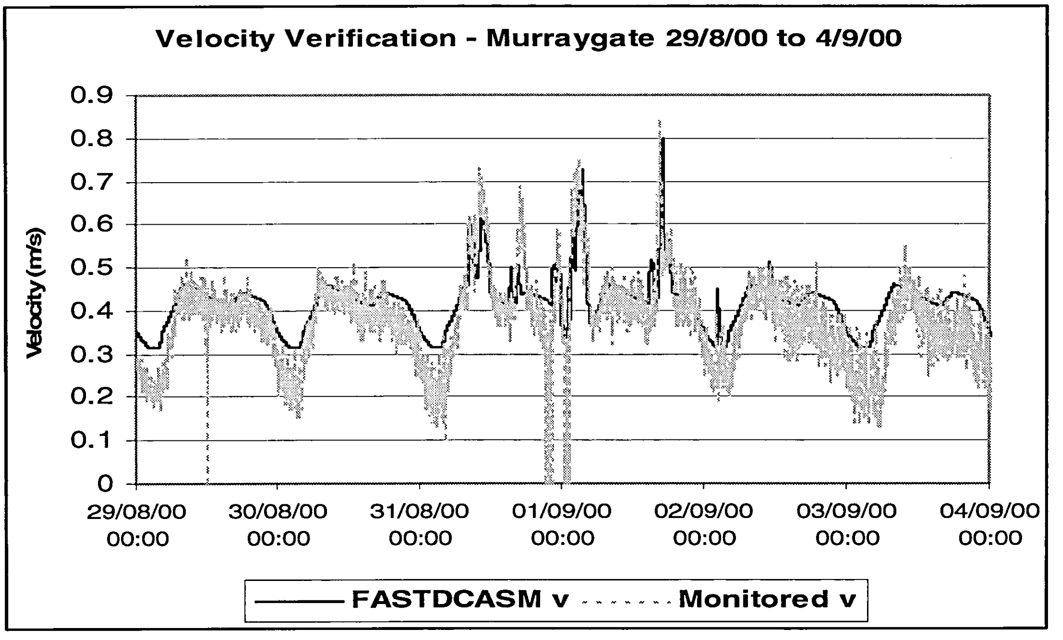


Figure 4.42 - Murraygate velocity verification

Figure 4.38 to Figure 4.42 shows the results from these verification simulations. It should be noted that the largest storm peak flow rate is overpredicted by approximately 30%, with dry weather flows generally well represented. The closeness of the match for dry weather flows is expected, as dry weather profiles were calibrated at each calculation point.

Depth of flow is represented well during dry weather flows with some diversion evident at the lowest flows. During these times the depth is underestimated by approximately 20 %. It is believed that this is a result of the sensitivity to section shape at these low depths. The coarse method of characterising flow depths and velocities is limited, as clean pipe figures are first determined, and then adjusted for the presence of sediment. The use of a full solution hydraulic model would allow the sediment effects and section changes to be represented earlier in the hydraulic calculation and would reduce these sensitivities. However, at the time of development, commercially available full solution hydraulic models did not offer the capability or flexibility for long-term flow and rapid data generation.

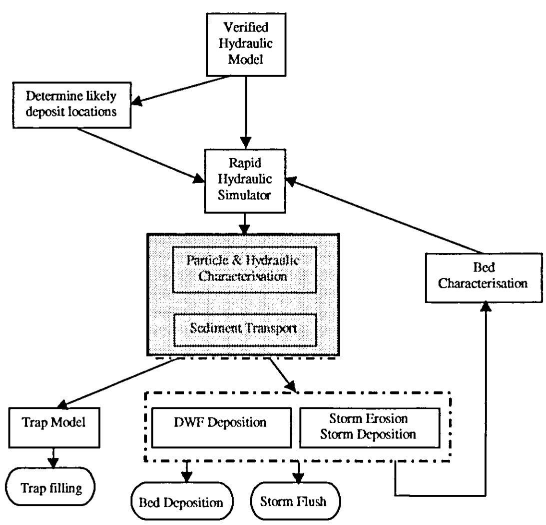
Peak depths are represented well, although it is evident that the measured dry weather flow depths temporarily reduce during the interim period of the three storms. An analysis of the flow data does not indicate any sensible reason why an actual reduction in depth would occur at this time, as during this period, increased flow depths would be expected as a consequence of higher infiltration and baseflows. An examination of the local pipe conditions indicates that the roughness and gradient of the pipe would not allow supercritical flows under all of the observed inputs. The decrease in depth is therefore believed to result from a malfunction of the logger such as the masking of the sensor by sediment deposited during the storms.

A similar pattern is shown for the velocities with excellent representation of measured results with the exception of the very lowest flows where an overprediction of 30% is experienced.

Although these simulations produce results well within the boundaries of acceptability for hydraulic modelling, the periods of low flow must be accurately represented for the purposes of sediment deposition modelling, as it is at these times that daily deposition takes place. These issues were therefore revisited during the development of the sediment models and are discussed further in Section 4.11.

4.6 Sediment Transport Model

4.6.1 Model Conceptualisation



At locations where sediment deposition has been shown to be a potential risk, it will still be necessary to gauge the extent of that risk and devise suitable strategies to manage the problems. In order to determine this, it is necessary to achieve accurate estimates of the sediment transport rates at these key locations. These data can then be used as inputs for determining the rates of sediment deposition, the rates of sediment trap filling or for determining the pollutant load present in the flow at any given time.

Significant research has been carried out in recent years in the area of sediment transport in sewers. Much of this research has focussed on finding improvements to existing relationships but still using traditional techniques of defining and modelling particle movement.

The failure of sediment modelling in urban drainage has previously been put down to a number of limitations (Verbanck et al., 1994; Berlamont & Torfs, 1996; Jack et al., 1996):

1. Field verification is difficult as a result of difficulties in the accurate measurement of inputs and outputs;
2. Traditional methods have been found to be extremely sensitive to changes in particle characteristics;
3. The range of particle characteristics found in sewerage systems, both spatially and temporally cannot be represented by traditional approaches;
4. The development of most techniques at present tends to concentrate on single particle sizes and density;
5. Cohesive effects are generally ignored.

It was the aim of this part of the study to provide sediment inputs to each of the deposition models (trap & pipe) under the wide range of flow and sediment conditions experienced in combined sewers.

Although the area of sediment transport in sewers provided no shortage of literature, this posed its own particular problems. Most notably: Which relationship should be used? An industry guidance manual, which attempts to provide engineers with an assessment of the most popular sediment transport relationships, is CIRIA Report 141 "Design of Sewers to Control Sediment Problems" (Ackers et al., 1996). This was used as an initial template for the assessment of the various techniques.

A major achievement made in the CIRIA report is the subdivision of the methods according to the conditions under which they were developed. In the report, this is done by determining if the relationship is suitable for modelling transport over a sediment bed or over a clean invert, and if the relationship is used to represent bed-load or suspended load. This approach was extended within this study to categorise relationships according to the regime in which they were developed (e.g. laboratory based, field based, physical model, empirical model, granular, cohesive, particle sizes, and particle types).

It was clear from this work that no current single model was capable of modelling the range of particle sizes, particle types and flow conditions that are experienced in combined sewerage systems. Records of field observations in the UK and France were therefore utilised to determine the different transport phases which may or may not be present at any given time. Within this study, three types of solid transport were differentiated with regard to the type of material likely to be transported near the pipe invert (as this material will contribute most significantly to deposition). These transport modes are broadly defined by the hydraulic regime in which they take place:

- Average dry weather flow – a mixture of fine granular and organic matter transported principally in suspension but with a bed load element present for larger granular material;
- Low dry weather flow – observed in the early hours of the morning or at locations with very low velocities. Corresponds to Arthur's "near bed solids" (Arthur, 1996), Verbanck's "dense undercurrent" (Verbanck, 2000) or Ahyerre's fine suspension (Ahyerre, 1999). Material largely comprises organic material of low density and toilet paper derived particles.
- High velocity flow – principally traditional granular bed-load material moved under high flow velocities and turbulence. This material is most usually observed under storm conditions.

Deposits of each of these characteristics were observed in the cryogenic cores taken from the sediment traps observed in this study (Chapter 3). It is clear from this that even for a single location, traditional methods of describing and predicting sediment transport are inadequate. The implications over an entire catchment are even greater, with the "global" selection of the particle used supposedly defining the level or predicted deposition at all locations throughout a catchment. In reality, the particles with the greatest settling velocity will preferentially settle first with progressively finer and less dense particles settling downstream (should suitable conditions exist). The characteristics of the sediment at any point in the system are in fact controlled by the hydraulic conditions upstream of that point.

It is therefore proposed that the user of any drainage sediment transport model should have only limited control of particle characteristics and should only set a range or size distribution for sediment inputs (DWF, storm and existing deposits). The remaining selection of particle characteristics should be calculated within the model as the sediments are routed through the system. This results in a relatively complex model requiring different sub models to calculate sediment transport rates under each of the three regimes described above (Figure 4.44). However, it also results in a simpler user interface and removes a large potential for the "force fitting" of a sediment transport model using unrealistic particle sizes and densities.

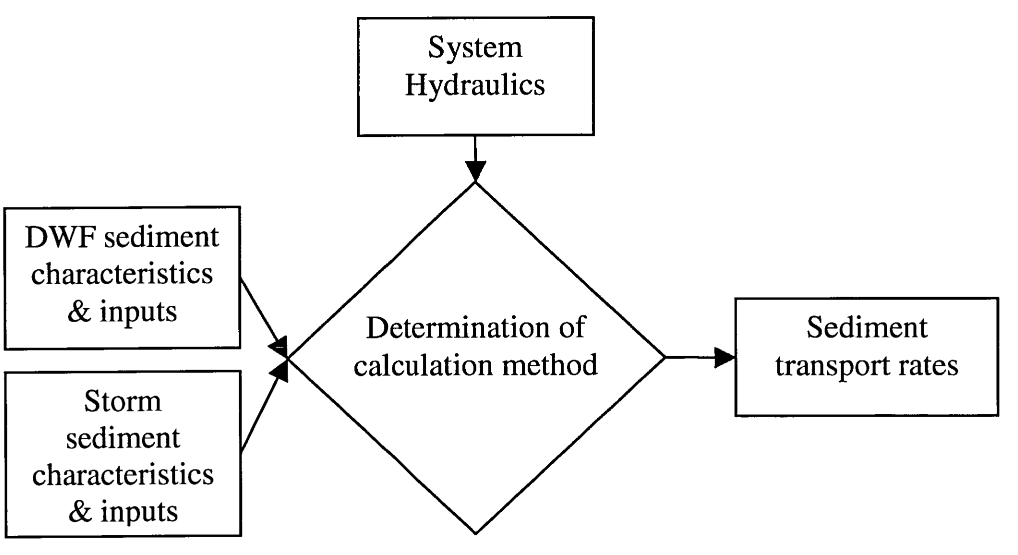


Figure 4.44 - Improved sediment transport model structure

4.6.2 Selecting Appropriate Relationships

The CIRIA 141 report (Ackers et al., 1996) was used in conjunction with previous research findings to select numerical relationships suitable to represent the three hypothesised transport conditions.

The relationship of Ackers and White was initially developed for use in river studies from rectangular flume experiments. However, since this time a number of studies have been undertaken to modify and apply these relationships to closed conduit and sewer systems (May, 1982; Ackers, 1991). Consequently, these studies have resulted in the testing of Ackers and White transport relationships over a range of particle sizes from 0.04 mm to 132 mm. This equates to a range of Ackers' Dgr value of between 1.1 and 132. It should be noted however that all of these tests have been concerned with mineral material only. Its development as a total-load approach also suggests its suitability to be used to determine concentrations when the range of particles present in the flow it is at its largest. The Ackers and White relationships were therefore used to represent the granular storm solids.

The guidelines of CIRIA 141 recommend the use of May (1993) to represent bedload at the limit of deposition. A review of the data used to develop the May equations was carried out. This assessment then compared the characteristics of these solids to those found in UK field studies. It was concluded that over the sizes of particle present during average dry weather flow, that the method of May was suitable.

The selection of an approach to represent low flow solids movement was more difficult as a result of the dearth of models for this "transport mode". The significance of these near bed solids has only relatively recently been highlighted, with continuing studies underway in the UK and France. Essentially the choice of method came down to just two.

1. Arthur (1996) – an empirical relationship applied using the current and historical hydraulic conditions in addition to basic assumptions of near bed solid properties (e.g. overall bulk density).

2. Verbanck (2001) – a semi-deterministic approach which considers the material as a fine suspension characterised by a definable profile. Due to the recency of this method, its application was largely untested.

It was feared that the use of the Arthur model could restrict applicability of the model as the relationship had been developed solely in Dundee's sewers. The empirical nature of the relationship is likely to lead to a site specific performance. As the Verbanck model was largely untested, historical data sets were used under low flow conditions to test the applicability of the method to such flows.

4.6.2.1 Profile Model Testing

Previous investigations into the movement of sewer solids have suggested that traditional descriptions of 'bed-load', derived from predominantly fluvial studies, are not strictly applicable in the cases of sewer sediments and hydraulics. It has been noted that in certain hydraulic conditions, the material moving near the pipe invert is highly organic and of comparatively low density (Arthur & Ashley, 1997; Ristenpart, 1995; Verbanck, 1995). This material and its form of transport have been given many terms (e.g. fluid mud, dense undercurrents, near bed solids). It is important however, that the term used should avoid misleading assumptions being made regarding the material or mode of transport.

Work carried out in Dundee has proposed that the term 'near bed solids' be used as a general term for the highly organic sub-layer (Arthur, 1996). However, this work has concentrated on the site calibration of traditional bed-load flume studies only, and does not consider the movement of these solids as a suspension. Alternative research has indicated the potential of using parabolic relationships in order to predict suspended solids profiles (Skipworth, 1996; Ristenpart, 1995). Development of this approach has been carried out in Brussels and involves the use of a two layer model, split at the interface of the bottom quarter and top three quarters of flow. The distribution of sediments in the top three quarters of the flow column is governed by the following law (Verbanck, 1995):

$$\frac{C_y}{C_{a*}} = \exp^{\eta(1-\frac{y}{a*})}$$ **Equation 4-3**

The suspended solids distribution in the bottom quarter of the flow column is given by (Coleman, 1982):

$$\frac{C_y}{C_{a*}} = \left(\frac{y}{a*}\right)^{-\eta}$$ **Equation 4-5**

As no applications of this approach to sewer sediment transport were known at the time of the study, the method was tested for applicability to this area of work. The procedure has therefore been applied to data collected during the Arthur study of near bed solids (Arthur, 1996). It should be noted that this approach was subsequently tested on wider data sets by Verbanck (Verbanck, 2000).

4.6.2.1.1 TSS Profiles - Dundee

An extensive programme of site investigations was undertaken during the Arthur study of near bed solids (Arthur, 1996). Much of this work focused on a sampling station located on Dundee's interceptor sewer. An artificial sewer was constructed within the city's Constable Street invert trap, to allow sampling and monitoring of the solids transported. Part of this study involved the measurement of total suspended solids (TSS) variation with flow depth. This was undertaken by obtaining samples, via small diameter tubes at predetermined positions in the flow column. The positions selected were chosen on the basis of the likely total depth of flow throughout the sampling durations, and were namely: 5 mm, 150 mm, 300 mm and 450 mm above the invert level.

Multi-depth sampling was carried out in this way on four separate dates, in dry weather, on 28/6/95, 10/7/95, 1/8/95 and 9/8/95. These data were used to produce a mean TSS profile for the Constable Street site (Table 4.14).

Sample Distance Above Invert (cm)	TSS (mg/l)
0	278
15	234
30	206
45	188

Table 4.14 - Average TSS Profile: Constable Street

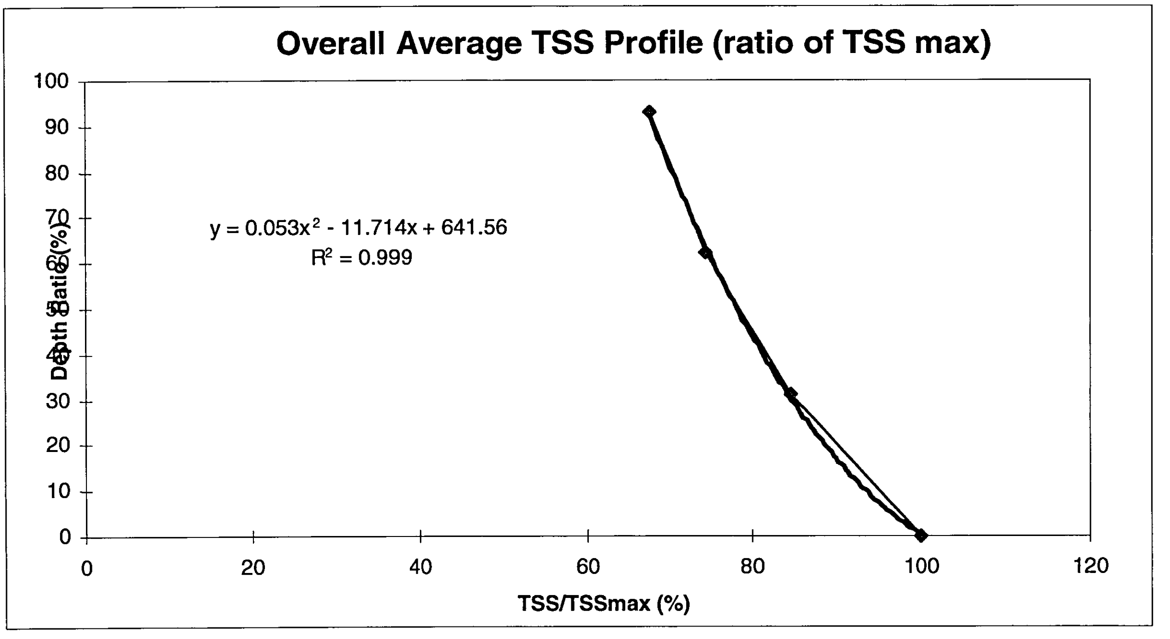


Figure 4.45 - Average Measured TSS Profile: Constable Street

The averaged data shows a definite TSS profile to exist (Figure 4.45). The profile in Figure 4.45 shows the normalised profile with both axes expressed as a percentage of the maxima. However, the profile was noted to be less pronounced than that observed by Ristenpart (1995) who obtained samples of near bed solids moving 10 mm above a deposited sediment bed using small bore samplers (Arthur, 1996). It was not known however, to what extent the sampler obtained samples of the near bed solids and what proportion came from the deposited bed. It should also be noted that problems were encountered during the sampling at Constable Street, in that blinding of the lowest sample tube frequently took place. This resulted in the blockage material acting as a filter, thus introducing the possibility of a reduced measured TSS concentration.

As an initial test of the profiling functions, these multi-level samples were used to provide reference levels for the application of the TSS profile relationships.

4.6.2.1.2 Verbanck / Coleman Profiles

A key aspect in the use of either Equation 4-3 or Equation 4-5, is the choice of reference level, a*, and reference concentration, C_{a*}. In order to test the effects of this, profiles were calculated using each of the top three TSS measurement points. The lowest point was omitted due to the sampling difficulties experienced in this area and calculation instabilities as the depth approaches zero. For calculation purposes, the base of the profile was assumed to be at the centre of a sampling hose resting on the pipe invert (i.e. 5 mm).

4.6.2.1.3 Verbanck Equation Parameters

As extensive data collection had taken place, measured values were available for almost all of the parameters required for the use of the procedure.

- **C_{a*}** - Concentration at reference level a_*. As profiles had been measured on various occasions, a mean concentration for each level was adopted.
- **κ** - A value of 0.4 was assumed for von Karman's constant.
- **w_s** – Settling velocity. Settling velocity tests were carried out for each of the depths sampled. Actual w_{50} values were therefore used. It was noted however that significant differences existed between the top three, and bottom samples. Settling velocities in the lowest quarter were observed to be approximately three times those of the upper three quarters. Consequently, tests were also carried out using a separate w_{50} for the lower quarter of the flow.
- **u_*** - Shear velocity. As velocity profile measurements had not been taken, the average shear velocity experienced was calculated using the boundary shear (τ_0) at the Constable Street site as:

$$u_* = \sqrt{\frac{\tau_0}{\rho_w}}$$ **Equation 4-7**

4.6.2.1.4 Profiling Results

Profiles were calculated for each reference level, initially using the mean w_{50} of all sample points of 1 mm/s. The resulting profiles can be seen below in Figure 4.47. It can be seen that the upper three-quarters of the TSS profile is best represented using the 300 mm reference level. This is expected as the selection of the centre value will always minimise errors in a near linear relationship. The relationship was found to marginally under-predict TSS concentrations in all cases. This was most severe when using the 150 mm reference level. This occurs as a result of a slacker gradient of profile if larger values are used for a* (thus decreasing the rate of change of the exponential power term).

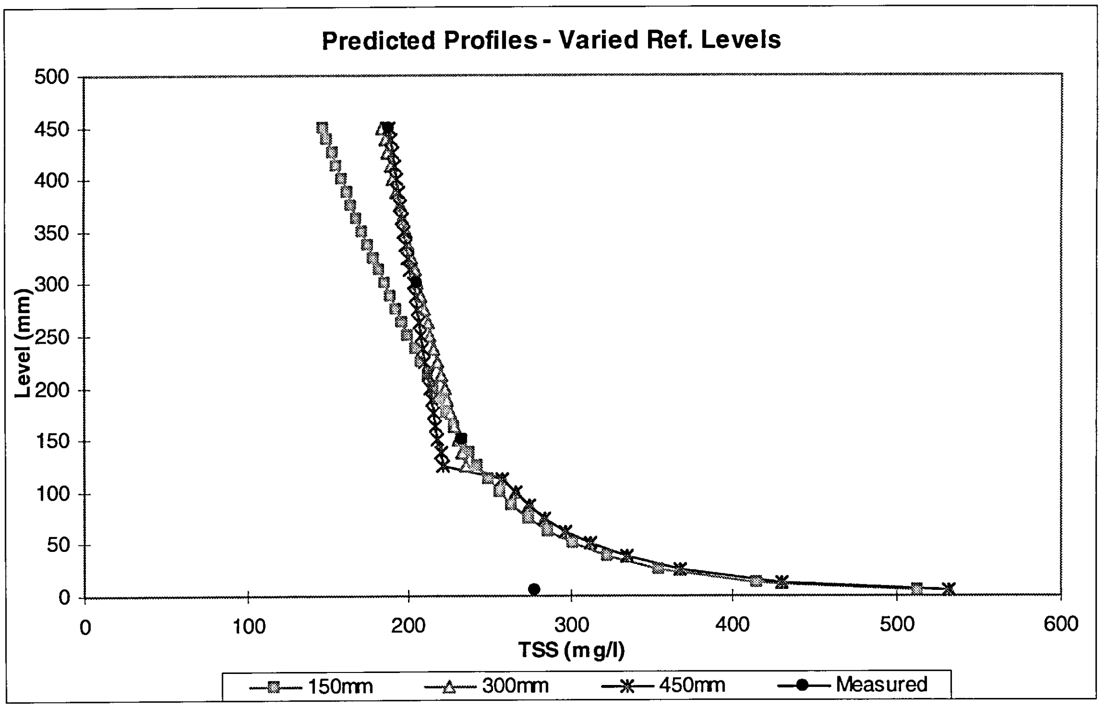


Figure 4.47 - Calculated Profile Variation With a*

The sediment concentrations of the lower portion of the flow are governed by Equation 4-5. In this simpler relationship, the effects of the choice of reference level are minimal, resulting in very similar curves for each reference level. The trend of prediction differs markedly from the upper regions of the flow column, as values are now generally over-predicted. It is believed however, that this is most likely a result of the sampling difficulties encountered in the near bed area. Therefore, the true extent of the accuracy of fit in this area cannot be tested using the profile data alone.

Further tests were also carried out using a more realistic figure for the settling velocity of particles in the lower quarter of the flow. The mean sampled w_{50} in this area of 3 mm/s was used. The resulting profiles (Figure 4.49) are identical in the upper three quarter region, but very different in the lowest area of flow. The curves approach asymptote much more quickly, producing a much more pronounced 'foot shape' to the profile. Although peak concentrations at y=5 mm are increased by around a factor of 7 (depending upon the profile considered), the real effect on the quantity of sediment represented is best evaluated considering the areas under the curves (i.e. total load per unit width). When using the increased settling velocity taken from the lower sampling point (3 x the upper), the area beneath the curve is increased by a factor of 2.7. As the lower curve is asymptotic to the x-axis, the point at which it becomes invalid is important regarding calculation stability and the total mass of solids present in the flow. The effects of this are investigated further in the calculation of NBS transport rates.

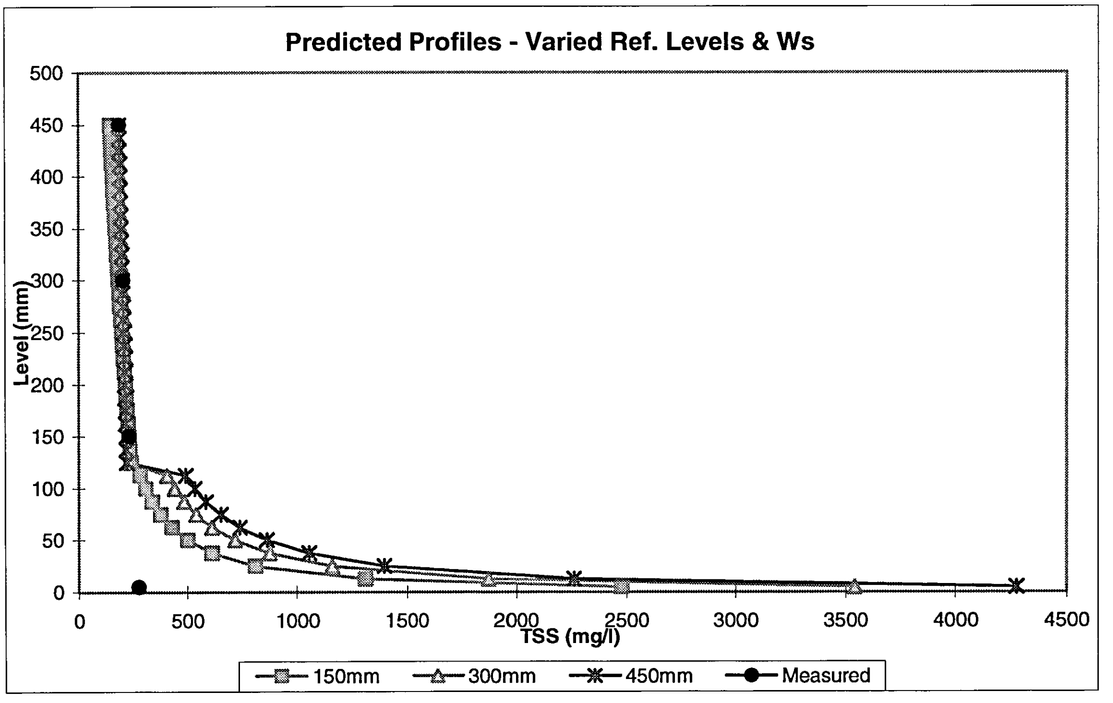


Figure 4.49 - Predicted TSS Profiles: Using W_s Upper & Lower

4.6.2.1.5 Near Bed Solids Transport Rates

Problems associated with the sampling of solids moving near the bed, or solids too large for the small diameter sampling hoses, have restricted the validity of the observed profile data. It is believed that a far larger concentration of suspended solids exists next to the bed than has been indicated by tube sampled values. In order to establish this, trapped samples (taken as part of the Arthur study) were used as an estimate of the rate of solids transport near the bed. As an average TSS profile had been used to determine calculated transport rates, a mean measured transport rate was used for comparison purposes. Transport rates were determined via the collection of 'bedload' samples using small invert traps. Assuming that these traps are able to collect all material moving near the bed over a given time, a reasonable estimate of the transport rate can be made. This procedure gave an average NBS transport rate of 238.7 mg/s.

The lower quarter of the flow was assumed to be the area through which the near bed solids were transported. This zone was selected in line with the assumptions made for the determination of the TSS profile (Ashley & Verbanck, 1997). The calculated

TSS profile within this area is defined by the Coleman equation (Equation 4-5), using a reference level of 300 mm (as measured during sediment profiling) and a lower flow level settling velocity. The assumption of the lower limit of the profile was retained from the previous investigations (5 mm).

With the TSS profile to be used defined, the average rate of solids transport was determined, using the following procedure;

1. The area under the TSS curve was determined using definite integration. This was carried out for subdivisions of depth to be used in stage 2. Strips of 25 mm thickness were used, with a top strip thickness of 12.5 mm to account for flow geometry.
2. The total area under the profile curve represents the instantaneous concentration per plan m^2 of flow. In order to determine actual loads, it is necessary to multiply this value by the flow width. As the flow width varies with depth, definite integration was also carried in plan to determine the sediment load. The average width of each strip was determined using a polynomial curve-fit of depth versus width for the pipe section.
3. The load per metre run (established by step 2) was multiplied by the average shear velocity (u_*) for the Constable Street site during the sampling period. This gives a rate of NBS transport that can be compared with the mean measured rate.

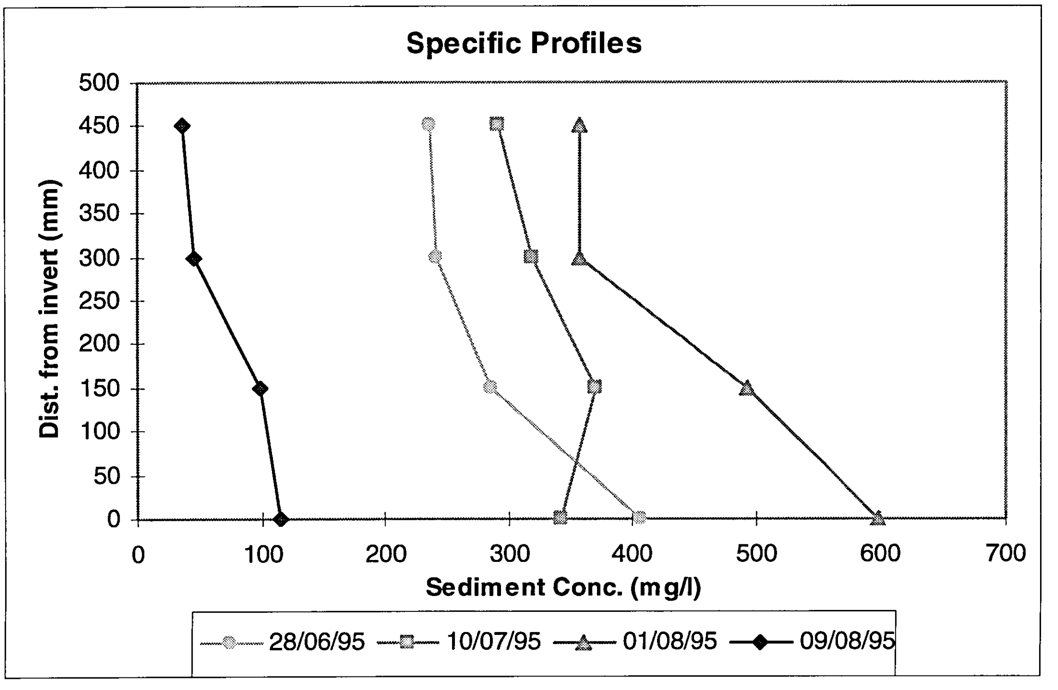
The results of this analysis are summarised below in Table 4.15.

Observed Mean Trans Rate	Predicted Mean Trans Rate	% Difference
239 mg/s	283 mg/s	+18.58 %

Table 4.15 - Summary of Results

The results from this initial investigation suggested that the procedure can provide comparable results using very few assumed parameters (given a high degree of data collection). However, as the inputs used were taken from averaged data sets, only general conclusions can be drawn. The results also suggest that the assumption regarding the relative accuracy of data collected in the near bed region (via the small diameter hoses) is correct and that a substantial portion of the solids are not sampled.

In order to further test this, the transport rate was calculated using the average measured profile (Figure 4.45). The resulting transport rate of 102 mg/l gives a discrepancy of -57% of the mean measured bed transport rate for the same site.



Prior to sensitivity analysis being carried out, the procedure was tested against specific data events. However, as profile and transport data collection were not carried out simultaneously, certain assumptions regarding the NBS transport rate and flow velocity had to be made. TSS profiling data collected at a given time of day (dry weather conditions) were used to calculate a corresponding NBS transport rate. This transport rate was then compared with a measured NBS transport rate taken at a similar time in the dry weather diurnal cycle (±20 mins). Consequently, differences may exist between the data sets, as they are not concurrent.

For each measured profile (Figure 4.51), a calculated profile was determined using the TSS_{300} (TSS concentration at a height of 300 mm), and transport rates calculated for both calculated and measured profiles. These data were then compared with bed trap NBS total transport rates, sampled at the same point in the dry weather diurnal cycle.

Figure 4.51 - Individual Profiles Used to Calculate Transport Rates

Date/Time	Bed Trap Measured Rate (kg/sec)	Calculated Rate (Coleman Profile)		Calculated Rate (Measured Profile)	
		kg/sec	% diff.	kg/sec	% diff.
28/6/95	4.892×10^{-4}	4.245×10^{-4}	-13.2	1.261×10^{-4}	-74.2
10/7/95	2.008×10^{-4}	4.139×10^{-4}	+106.1	1.466×10^{-4}	-27.0
1/8/95	4.564×10^{-4}	4.923×10^{-4}	+7.9	2.353×10^{-4}	-48.45
9/8/95	0.847×10^{-4}	0.612×10^{-4}	-27.8	0.362×10^{-4}	-57.19

Table 4.16 - Comparison of Specific Transport Rates

Although the measured and calculated rates of transport are not directly comparable (Table 4.16), it can be seen that in general that the Coleman profile method gives results within the standard accuracy parameters of alluvial sediment transport methods. The exception to this occurs when using the 10/7/95 profile where the calculated rate of transport is more than double that measured. It is believed that this is a result of the transport rate samples and TSS profile samples being taken at different times of the day. A comparison of the transport rates based on measured profiles supports this assertion. The absolute percentage difference for this particular event is significantly lower than any of the others. As the transport rate is more significantly under predicted in all three other cases, it is assumed that these data sets are not directly comparable.

4.6.2.1.6 Sensitivity Testing

As a result of the limited quantity of data collected during the Arthur study and the low number of input parameters required in the procedure, sensitivity testing could only be carried out on four main data items (original values shown in brackets):

- C_{a*} - reference concentration (206 mg/l);
- w_s - settling velocity (3 mm/s);
- u_* - shear velocity (0.0108 m/s);
- Lower limit of calculated profile (5 mm above invert).

As the reference concentration (C_{a*}) is used purely as a constant of multiplication, its effect on the overall transport rate is directly linear.

Figure 4.53 (below) shows the effects on NBS transport rate, of changes in the settling velocity used. It can be seen that the procedure is of high sensitivity when the settling velocities are in excess of a 25 % increase (3.75 mm/s), and that below this value, the relationship shows a much lower rate of change in transport rate with changes in settling velocity. This may be of high importance, as a variety of methods for the determination of settling velocity exist. For example, the UFT method (see Appendix C) has been shown to give higher settling velocities (between 2 and 30 times), than the Aston settling velocity test (Arthur, 1996). The settling velocity influences the concentration profile via the power term η (Equation 4-9).

$$\eta = \frac{w_s}{\kappa u_*}$$ **Equation 4-9**

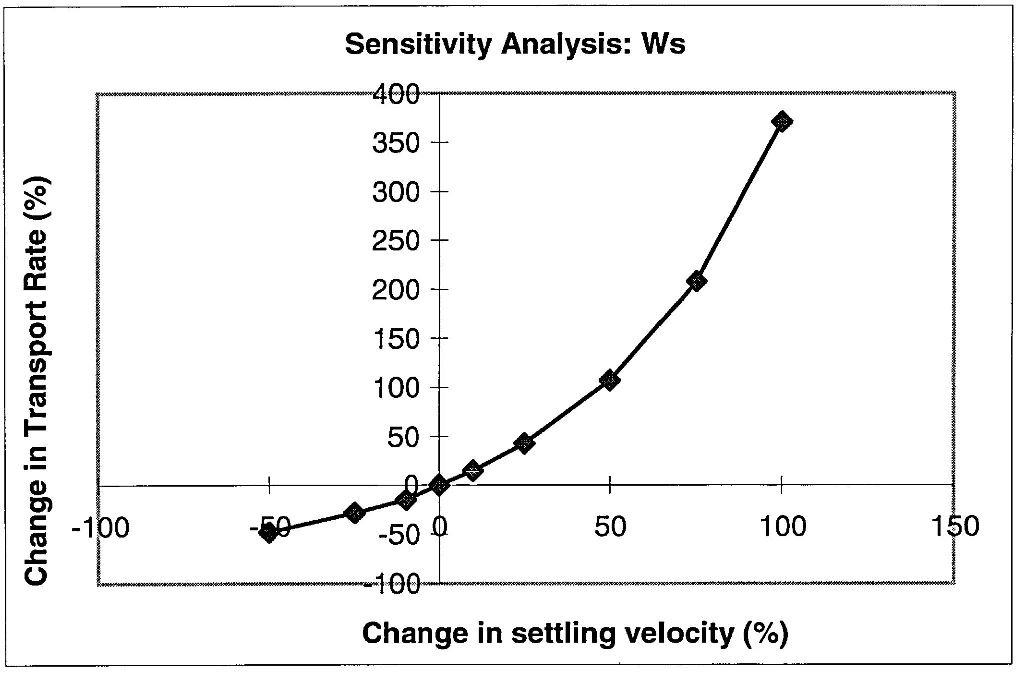


Figure 4.53 - Settling velocity sensitivity

The alteration of u* was found to have a more complex effect on the calculated transport rate (Figure 4.55). This derives from the two ways in which the shear velocity is used in the procedure. u* is used to determine the power term η of the concentration profile relationships (Equation 4-3 and Equation 4-5), and is also used as a direct multiplier to give the transport rate.

Consequently, as the shear velocity is reduced, the TSS concentration is increased, but the velocity of the particles is also reduced. The net effect of this varies depending upon the value of shear velocity chosen. At low values of shear velocity, the high TSS concentration produced, dominates the overall transport rate, which is increased. Increases in the shear velocity up to approximately +75%, produce a slight reduction in the total transport rate, as a mixture of reduced concentrations and increased velocities combine to marginally reduce the overall transport rate. Further increases in shear velocity result in an increased transport rate, as the particle velocity becomes the dominant factor in the determination of NBS transport. The above results suggest that even in the case of reduced hydraulic survey, an overestimate of the shear velocity (up to +150%) could be used to achieve reasonable results.

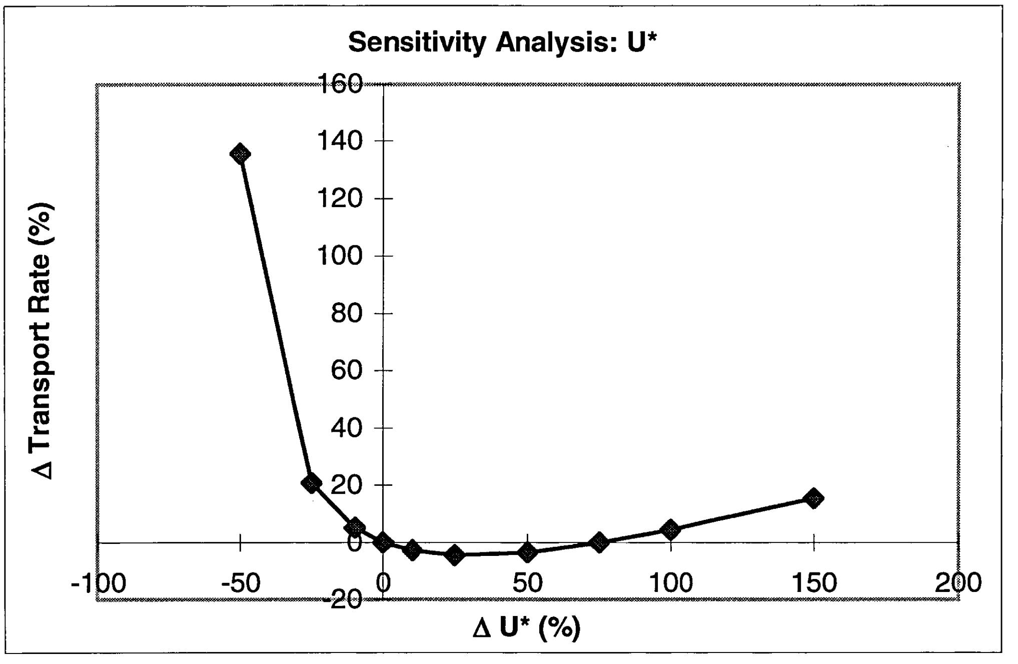


Figure 4.55 - Shear velocity sensitivity

Prior to these investigations, it was believed that the selection of the lower limit of the TSS concentration profile would be a crucial element in the determination of the transport rate. As the profile is asymptotic to the pipe invert, theoretically, an infinite sediment concentration exists immediately adjacent to this location. However, this clearly is not the case, and some sensible limit should be set for the extension of the profile. It is believed that this value should be approximately equal to the clean k_s value of the pipe considered. In cases where a deposited sediment bed is present, the bed roughness should not be used, as part of the bed material (contributing to roughness) may be in motion, and therefore forms part of the TSS profile. A reduced form of the bed roughness is advised.

Figure 4.57 shows the changes in transport rate with altered profile limit. It can be seen that the total transport rate is relatively insensitive to changes in lower profile limit, a +200% increase in the level of the profile limit resulting in a transport rate reduction of approximately 15%.

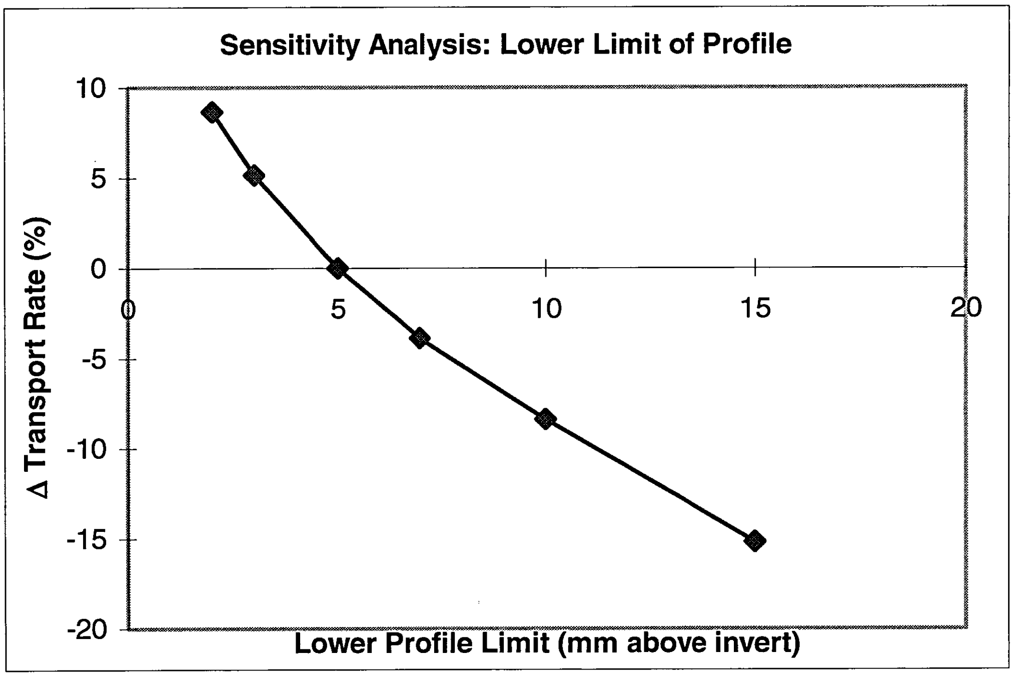


Figure 4.57 - Lower profile limit sensitivity

4.6.2.1.7 Profile Testing Conclusions

The use of TSS profiling techniques for the prediction of NBS transport rates has been shown (in the above cases) to provide a level of correspondence to observation exceeding that generally accepted in the field of sediment transport. Although the procedure may furnish researchers with an additional means to predict this mode of transport, its use is severely restricted due to the level of data collection required. Most notably, a TSS concentration at a fixed level is required, making the procedure applicable only to the analysis of existing systems or those for which characteristic profiles may be defined.

The investigations carried out, have highlighted possible recommendations for any future use (or study) of the profile method.

- The reference level a_*, should be located at a height of ¼ of the total depth above pipe invert. This will allow errors to be minimised in both the upper and lower areas and provide the most realistic representation of profile shapes.

- An over-estimate of shear velocity (u_*) is of reduced sensitivity compared to an under-estimate.
- The lower extremity of a calculated profile should be limited by the roughness of the pipe considered.

The above points should also reduce the level of data collection required, thus enhancing the methods' widespread use. However, three main questions require to be addressed before any specific conclusions can be drawn:

- Are the predicted material movements of the same 'type' (transport mode, physical characteristics) as the bed trap collected material?
- Will the simultaneous measurement of transport rate and TSS profile confirm/contradict previous data?
- What is the widespread accuracy of the calculated reference concentration?

Should these areas be addressed satisfactorily, the TSS profiling approach would provide an easily applied, alternative method of predicting NBS transport rates in sewers.

4.6.3 Defining Hydraulic Limits for Transport Modes

The relationships that were selected to be used in the sediment transport model are shown below (Table 4.17). Each of these methods is discussed in Section 2.7.3.

Flow Regime	Sediment Transport Method
Low dry weather flows	Verbanck (2001)
Average dry weather flows	May (1993)
Storm flows	Ackers and White (1991)

Table 4.17 - Selected sediment transport models

However, before applying these it is necessary to sensibly define the point at which each model becomes valid and should be used. To facilitate this, the regime under which each model was developed and calibrated was used to define the most applicable conditions. These conditions were then expressed using Ackers and

White's dimensionless grain ratio, D_{gr} (Equation 4-11). The ranges of validity were then used to identify parameters for each relationship.

$$D_{gr} = \left[g\left(\frac{s-1}{\nu^2}\right) \right]^{\frac{1}{3}}$$ **Equation 4-11**

Where: Dgr = dimensionless grain parameter
g = acceleration due to gravity (m/s^{-2})
SG = specific gravity
ν = kinematic viscosity (m^2/s)

Dgr range	Relationship	η range
0-3	Verbanck (2001)	0-1
3-15	May (1993)	1-3
>15	Ackers and White (1991)	>3

Table 4.18 - Valid model ranges

Table 4.18 shows the valid ranges estimated using the procedure described above. The D_{gr} method was used since the procedure has long been applied to sewer sediments. As an alternative to this, the limits of applicability were also calculated using the settlement parameter η (Equation 2-7) to account further for hydraulic effects.

It is clear from both of the approaches used to define model applicability (Dgr and η range) that the dominant defining factor is that of particle size. It was therefore decided that the particle sizes present in the flow should be determined by local hydraulic conditions, rather than (as has been done in the past) being imposed upon the calculation. This assumption of a single characteristic particle size has long been considered a major restriction of the broad applicability of sediment transport methods to sewer sediments. In order to address the complication of varying sediment density, a default value for mineral material was first assumed to determine the Dgr of the particle. Following this initial assumption, the assumed density was modified in line with the type of solids being represented. Under low flows and the

Verbanck equations a low density suspension prevails, resulting in an assumed specific gravity of 1.25 being used for the calculation of sediment concentration. Under the May equations for "average dry weather flows", a mixture of sanitary and granular solids is assumed to prevail, with a specific density of 1.75. For storm flows where Ackers & White relationships are used, a granular dominated specific gravity of 2.2 is used.

Figure 4.59 shows the uppermost layer of the Fraser sediment transport model that allows the appropriate method of sediment transport calculation to be selected. Inputs of shear stress and an assumed particle size distribution are used to activate the correct sediment transport routines. Within this model, the characteristic particle sizes are determined using the method shown in Figure 4.64.

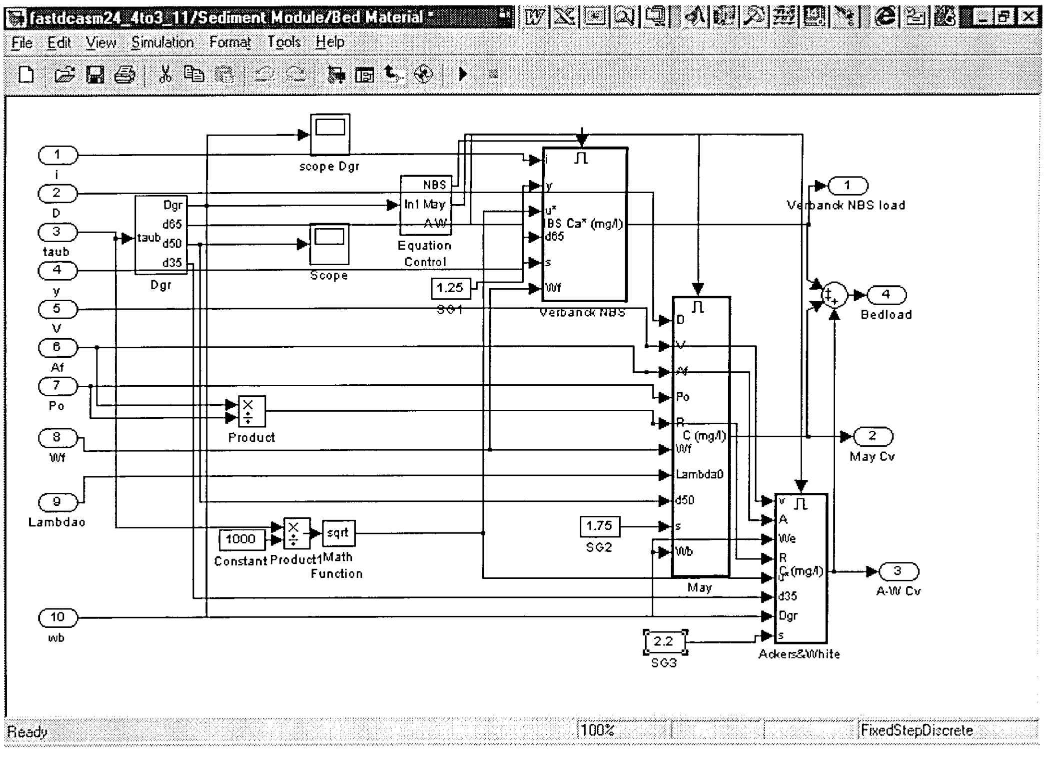


Figure 4.59 - Fraser model for method selection (uppermost layer)

4.6.4 Addressing Single Characteristic Particle Limitation

Sewer sediment particles have long been observed to vary greatly both spatially and temporally in characteristics. This variability has been highlighted as a severe modelling restriction by a large number of researchers (e.g. Jack et al., 1996; Ashley et al., 1999; Verbanck, 1994; Rushforth, 2003). As models within this thesis aim to address the prediction of solids at a variety of locations over prolonged durations, it is important that all likely solids types and flow regimes are included.

This problem was addressed at the outset of the study, with two principal solids types perceived to represent solids inputs. These were coarsely differentiated as dry weather and storm solids. It was hypothesised that, as foul inputs are found throughout the catchment, dry weather particles should exhibit relatively consistent characteristics spatially. A similar argument was used for storm solids as their inputs during runoff events are generally evenly distributed, resulting in similar input characteristics throughout the catchment. Whilst these assumptions have provided reasonable modelling results within the early parts of this study, the development of the sewer deposition model (and trapping model) facilitated the potential inclusion of more realistic particle characteristic calculations to the transport and deposition models.

It is useful from an operational perspective to be able to approximately characterise the type of material likely to deposit at any given location within the sewer network. This information allows maintenance managers to assess the potential problems associated with these deposits and also to devise the most suitable method of their removal and management. Within this context, three particle types were considered.

	Performance Issues	Removal Issues
Fine	May combine with larger particles to restrict flows. High polluting potential. Likely to be widespread.	Can be easily eroded but may develop cohesive strength. Unlikely to require frequent maintenance.
Medium	Restricts flows. Medium polluting potential.	Unlikely to be removed by flushing. May develop cohesive strength with fine material.
Coarse	Restrict flows. Likely to be localised.	Unlikely to be removed by flushing. May require frequent maintenance.

Table 4.19 - Operational division of particle types

In order to simplify the analysis, sediment density was assumed to remain constant, with storm and dry weather conditions considered separately. In this model, sediment inputs to the sewerage system are varied only according to weather conditions. During dry weather, only fine and gross solids are produced by the model, with a mixture of fine and coarse material used for model inputs during wet weather. This approach attempts to address part of the wide temporal variability of solid inputs but does not examine spatial variability. To do this, model processes must be used to modify the characteristics of the solids in transport. Within this proposed model, the principal method of accounting for this is to allow the largest particles to be removed from an assumed particle size distribution preferentially at a location of known sediment deposition. The particle size distribution exiting the deposit location is therefore modified and results in finer material being passed on. This approach is shown graphically in Figure 4.60. It can be seen that, not withstanding the effects of erosion and flushes, sediment particles are gradually "sieved" as they are transported through the sewerage network. This behaviour is particularly prevalent in coastal towns and cities which tend to have steep upper reaches of the catchment feeding down to relatively flat interceptor sewers.

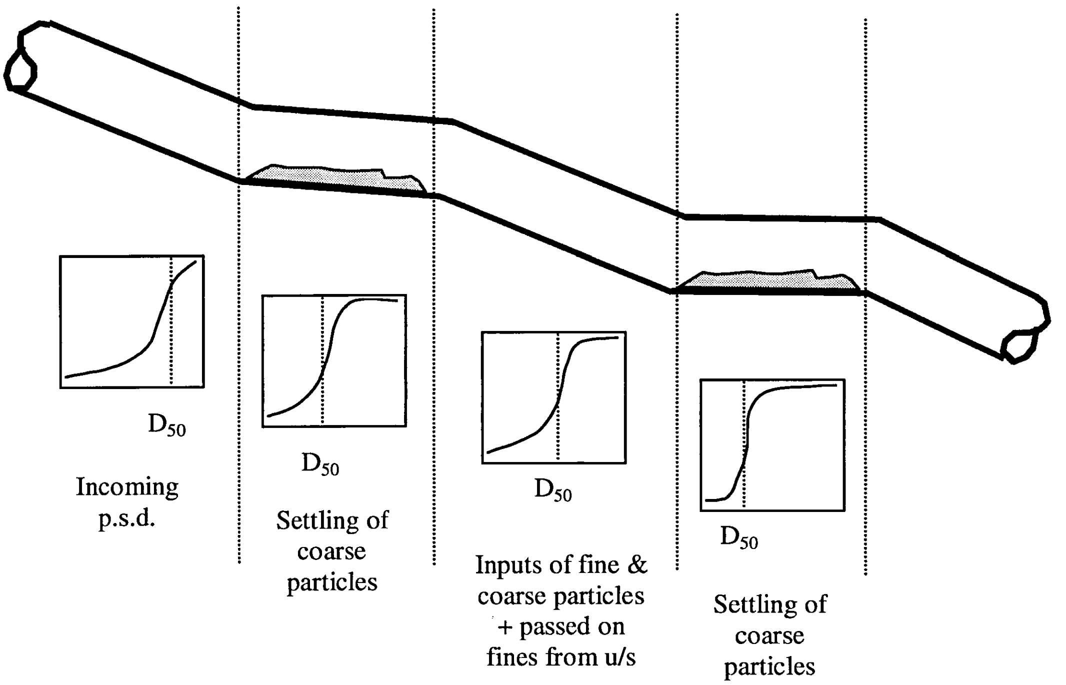


Figure 4.60 - Effect of sedimentation on particle size

The use of the procedure described above was tested at a local scale using limited data from previous Dundee City Centre studies and additional data collection. The initial test (and that most pertinent to this investigation) was to determine if the procedure could be used to predict approximate sizes of particle deposited at a particular location. Detailed knowledge of particle size distributions was used in conjunction with Shields' relationships (Shields, 1936) to determine the range of sizes removed from the incoming flow at three sites of deposition used in this study (Murraygate Interceptor, Claverhouse Trunk Sewer and Forfar Trunk Sewer). In each test, the maximum transportable size was calculated for a given hydraulic condition. A deposition factor was then also calculated for this hydraulic condition using the procedures described in Section 4.7, and applied to the particles larger than the maximum transportable size. This allows two new particle size distributions to be determined:

1. Settled particles
2. Particles passed downstream

Table 4.20 shows the results of this analysis. Bed samples were extracted as bulk "grab" samples. These were then dry sieved to determine the particle size distribution. In each case, peak dry weather flows were used in conjunction with Shields' criteria to determine the settled particle sizes. A good correlation is shown in each case, with Forfar showing the largest deviation (-0.07 mm). This is to be expected as much of the material was found to be of low density and therefore not suitable for the application of Shield's standard curve. Nevertheless, the approach is easy to apply and quickly allows an estimate of the particle sizes and likely characteristics and quality of any potential deposits.

Location	Observed Bed d_{50}	Predicted Bed d_{50}
Claverhouse Trunk Sewer	0.97 mm	1.1 mm
Murraygate Interceptor	0.47 mm	0.5 mm
Forfar 900mm Trunk Sewer	0.17 mm	0.1 mm

Table 4.20 - Predicted deposit particle sizes

The full testing of the applicability of the approach to determining the particle size distribution of materials in transport could not be carried out. It was hoped that significant differences between the particle size distributions of suspended samples as they arrive and depart from a depositing sediment bed would be detected. The limited attempts to record this did not reveal any statistically significant differences. However, the general correlation of the deposited and predicted particle sizes suggests the future potential of this approach to estimate deposited particle characteristics. It is further suggested that this information can be used to modify an assumed (or measured) particle size distribution to determine the particle size distribution following deposition. The combination of this approach and the intelligent selection of the transport relationships described in Section 4.6, results in the process described graphically in Figure 4.62.

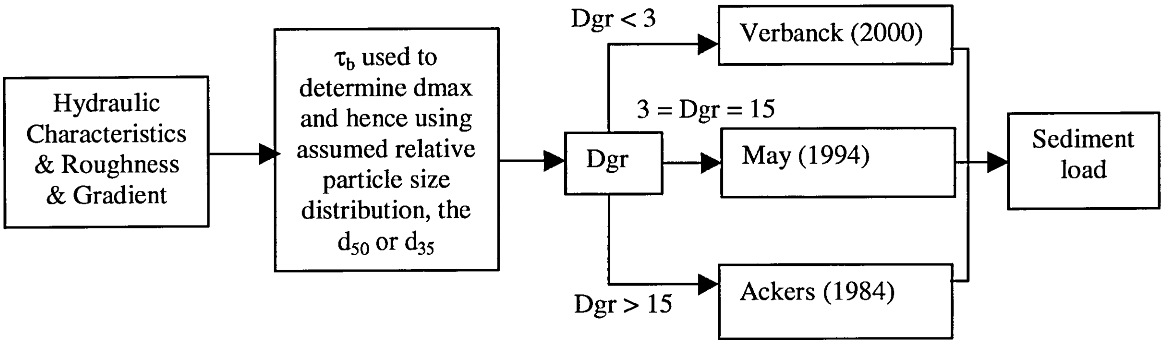


Figure 4.62 - Sediment transport relationship selection

The input variables used within the sediment transport model are shown below:

Variable	Source / Comment
Peak DWF	Measured or model data
Pipe slope and geometry	Measured or model data
Sediment density	Engineering estimate
fine sed %	Engineering estimate
med sed %	Engineering estimate
coarse sed %	Engineering estimate

Table 4.21 - Sediment transport model inputs

Figure 4.64 shows the portion of the Fraser sediment model for selecting the various characteristic particles (d_{35}, d_{50} and d_{65}) from an assumed particle size distribution. These particle sizes are then used as inputs to the sediment transport models.

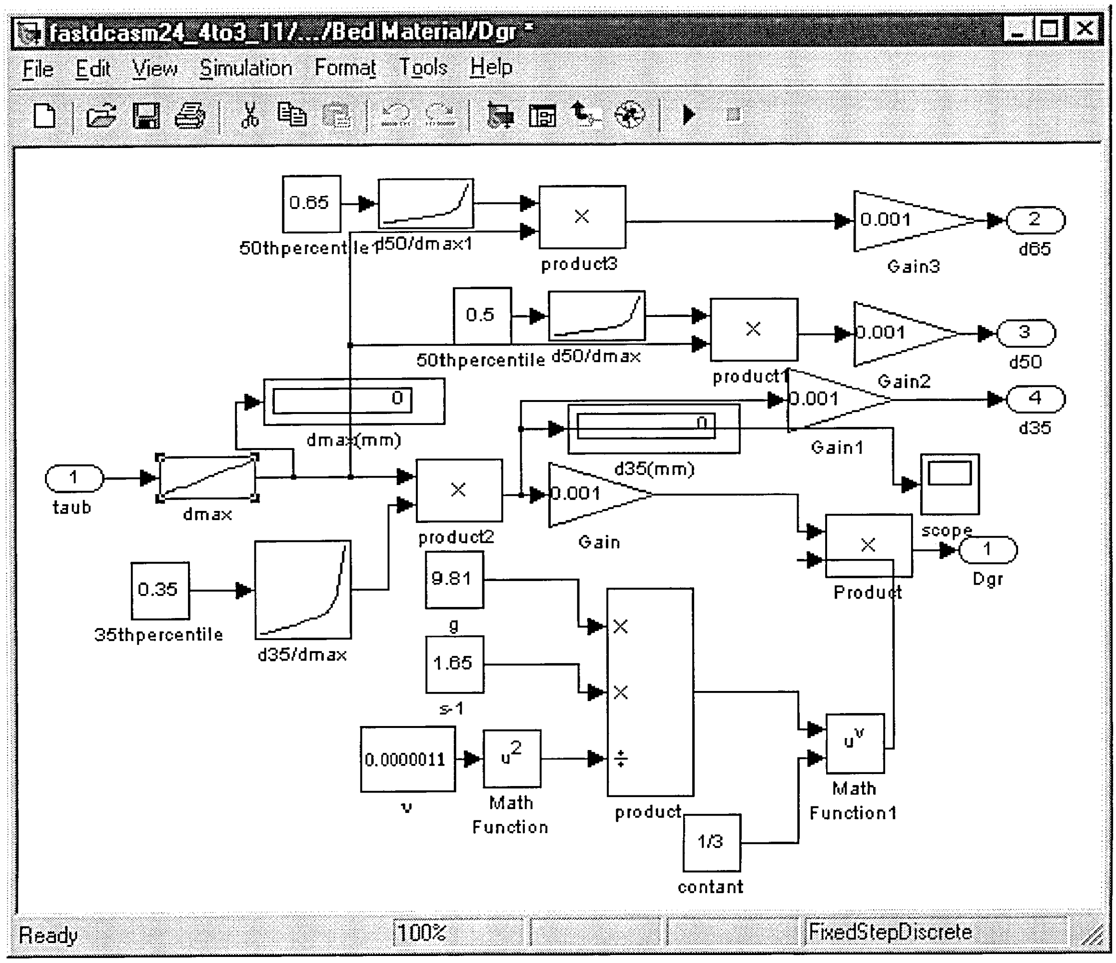
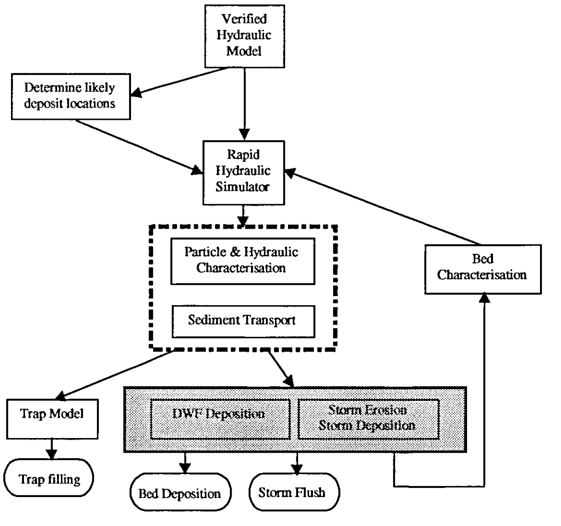


Figure 4.64 - Fraser model for size selection based on assumed particle size distribution

It is suggested that more detailed sediment transport models which attempt to route sediments through an entire catchment (e.g. InfoWorks or Mousetrap) will have an increased sensitivity to preferential settlement and its effects. It is therefore recommended that these phenomena should be investigated further for application to detailed models.

4.7 Sediment Deposition Quantity Model



In order to evaluate any potential problems posed by sediment deposits and in order to devise an appropriate sediment management strategy, it is necessary to achieve estimates of the likely quantities of sediment deposition. The sediment location model proposed previously can be applied to identify areas where a detailed time based simulation of flows, sediment transport, deposition and erosion can be implemented.

A review of previous approaches used to determine volumes and/or depths of sediment deposition was carried out (see Chapter 2). This review produced a shortlist of methods with suitable technical merit and simplicity of application. The method selected for further development was that of Pisano et al (1979). The procedure was developed by the USEPA and was selected as it had been verified at several locations (U.S. and U.K.), required only limited input parameters, contained few empirical variables and was simple to apply.

4.7.1 USEPA Model Limitations and Modifications

The procedure uses a ratio of applied shear stress to critical shear stress (along with empirical factors) to provide a deposition factor for each pipe in a network (for further details see Chapter 2). This deposition factor however, remains constant for any particular pipe as average flows are used. This assumption has been shown to be incorrect as experience of long term sediment build up in the UK and France has suggested that the deposition rate is extremely variable, with the sediment bed often reaching a state of equilibrium (Laplace, 1991; Ristenpart, 1995). This is believed to be related to the gradual increase of bed gradient as the ongoing process of sediment deposition and erosion shapes the bed. It is therefore essential that not only should the procedure use varying flows, but also the evolving bed gradient, width and roughness should be allowed to influence those flows.

A further limitation of the USEPA equation is that no explicit account is taken of the length of pipe considered. Clearly a longer pipe at a given gradient will have a greater capacity to retain more material than a shorter pipe at the same gradient (depending upon input concentrations). The USEPA method only accounts for this by the fact that a longer pipe will have a larger contributing population and hence larger solids loadings rate and deposition rate. As an alternative approach was being sought to represent the dry weather flows it was decided to base solids loadings for the pipe in question on a suitable sediment transport relationship. It was decided that a dry weather bed-load or near bed solids relationship should be used as this is the material type from which dry weather deposits are likely to form.

The issue of the length of pipe was addressed by revisiting the original data sets used to develop the procedure and modifying the deposition factors calculated, to provide a factor per metre run of each pipe. These were then used to determine an overall modified factor per metre run. An improved approach would be to subdivide each pipe with a number of calculation points. This would allow deposition rates to vary (and hence sediment bed levels) along the length of a pipe. This would require a full solution hydraulic model in order to represent the variation of hydraulics along the length of the pipe. However, as previously discussed, as a result of the limitations of HydroWorks QM, this was not an option at the time of development. The simplified approach to hydraulic modelling used within this study does not allow such complex calculations.

The format of the USEPA deposition percentage relationship was retained, as was the basis of the Shields and Hughmark relationships in its determination (Pisano et al. 1979). However, the factor of 40 was replaced as this initial value was related to the development of the relationship for per capita wastage rates (and hence used indirectly to represent pipe length). An additional factor was also developed within the relationship to account for the variation of bed width as the sediment is deposited.

The maximum influence of the sediment bed regarding deposition is assumed to occur when the bed width equals the maximum width of the pipe. At this point, the factor W_b/W_{max} equals 1. Clearly as the bed width approaches zero, the calculation becomes invalid as deposition will also tend to zero. For this reason it is suggested that a minimum bed width of ten times the particle d_{50} should be used for clean pipe conditions. This recommendation is made in line with sediment transport relationships determined for transport over deposited beds (Ackers et al., 1996; Nalluri et al., 1994).

Equation 4-13

$$Z = 0.889 \times \left(\frac{\tau_0}{\tau_c}\right)^{-1.2} \times \left(\frac{W_b}{W_{max}}\right)$$

Where: $Z =$ % of suspended solids depositing per m run of pipe
$\tau_0 =$ boundary shear stress given by ρgRs (where s is bed gradient)
$\tau_c =$ critical shear for mean particle size
$W_b =$ sediment bed width
$W_{max} =$ maximum pipe width

The resulting modified relationship is shown above (Equation 4-13). This relationship now allows a dynamic system to be modelled as a result of:

- Boundary stresses varying as a result of bed gradient changes;
- Flow conditions varying as a result of changed boundary conditions and section shape.

It should be noted that this model is used to represent deposition processes only and cannot be used to estimate erosion. As a consequence of the low number of variables used in the procedure, only limited calibration could be carried out. During the development of the model it was perceived that the empirical factor of 0.889 should be adjusted (although this was not required). However, during application of the model to other areas, calibration could be carried out through the variation of τ_c.

4.7.2 Modified Model Testing

The testing of the model using historic data was limited as a result of the dearth of long-term, detailed sediment deposit information. Initial testing was carried out using data collected as part of a previous sediment transport study (Coghlan, 1997). The study again focussed on the Murraygate Interceptor sewer in Dundee. Although not explicitly concerned with the collection of deposit data, a series of walkthroughs was undertaken to examine the structure and form of the sediment bed. These data are particularly valuable, as a "clean out" of the interceptor sewer was carried out during data collection. This allows the patterns of deposition to be assessed under greatly varying conditions.

The length of sewer used in the investigation was 175 m long, in a city centre shopping precinct. The area drained by the sewer was estimated at 340 hectares with a resident population of 14590. The average gradient of this sewer is 1 in 1446 (0.0007) and is virtually straight in alignment. The cross section of the sewer at its upstream end is 1.530 m high, with a maximum width of 1.415 m. Although the design section shape is egg type 2 at this point, detailed section surveys have revealed some deformation. The surveyed data were used for all calculations. There are major trunk sewer junctions at either end of the study length, with a flow control gate located at the mid point of the sewer.

Sediment depths were recorded during a series of 16 'walkthroughs' over a period of approximately two and a quarter years (June 1987 to September 1989). Consultation with the water authority revealed that few operational changes had been made to the system since this time.

From these data collection exercises, sediment depth profiles and time varying average sediment depths were produced. The sediment depth profiles indicated that certain locations where local disturbances to flow existed (turbulence created by the confluence with the Commercial Street sewer gate chambers at Horse Wynd and Peter Street junctions) were characterised by the shallowest sediment deposits. It was also apparent from these data that the sediment build up was greater in the upstream half of the study length, leading to an overall increase in the sediment bed gradient.

The study length of sewer was cleaned out in November 1989 resulting in a sudden reduction of average pipe sediment depths. The average sediment depths measured during the survey period can be seen below in Figure 4.65.

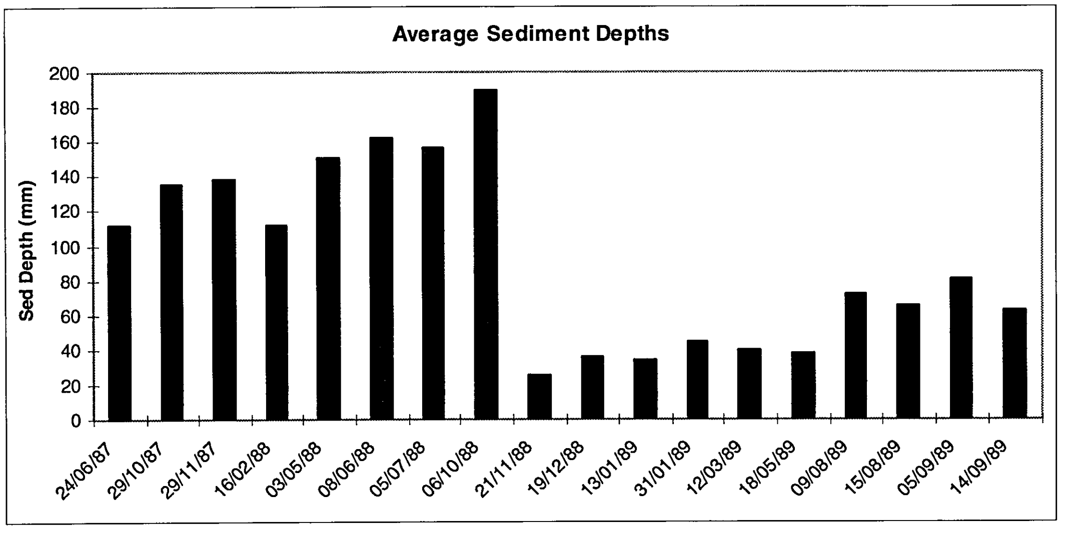


Figure 4.65 - Average Murraygate sediment levels 24/6/87 to 14/9/89

It is clear from Figure 4.65 that the deposition processes were continuing immediately prior to the clean out. There is therefore no indication from these data of the existence of an equilibrium level of sediment deposits. However, the operational experience of Scottish Water staff has shown that this did in fact exist at an average level of between 500 to 700 mm. The range of depths experienced for this pipe average is not known.

The average bed gradient along the length of pipe was also recorded during the survey. Figure 4.67 (below) shows how these changed with time. In general it was observed that as sediment levels increase, the bed gradient also increases. The rate of this change of bed gradient seems to reduce at the highest sediment depths. These observations support the assertions of Lin and Guennec (1996), that the rate of deposition is reduced over time by the gradual increase in bed gradient as the bed develops.

The exception to this is seen following the clean out of the sewer. Assuming that the clean out of the sewer is total, a starting gradient of 0.0007 would be expected (as this is the gradient of the pipe). However, following the clean out, a reduction of gradient is observed as the deposits start to form

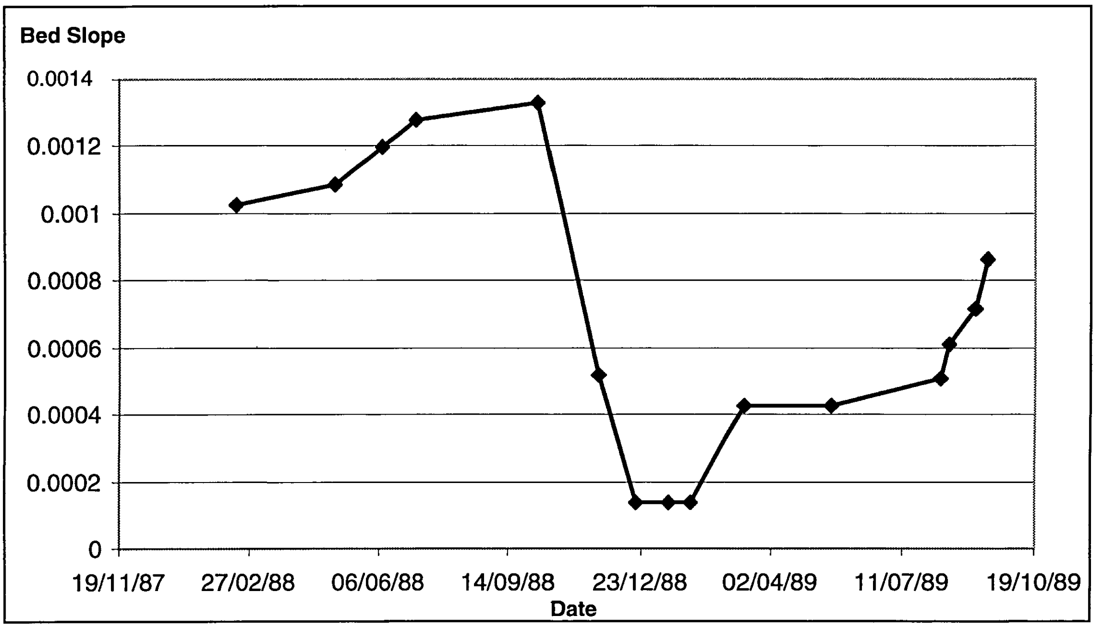


Figure 4.67 - Observed Murraygate bed gradient 16/2/87 to 14/9/89

The above observations were used to hypothesise the following routine of sediment deposition using the simple case of a straight, shallow pipe being fed inputs from a steeper pipe:

1. As sewage flows enter the pipe, the flow slows. The resulting reduction in sediment transport capacity allows the largest particles to settle at a distance from the pipe inlet controlled by the velocity of flow and settling velocity of the particles. Progressively smaller particles will settle downstream from this until the new sediment transport capacity of the flow is reached. This effectively

reduces the average pipe gradient as the deposits will not initially form at the pipe inlet.

2. The initial deposits will then serve as an obstruction to the flow and will slow the flow locally, immediately upstream of the initial deposit. The pattern of deposition will therefore work upstream from the initial deposits. At this stage the average gradient will therefore tend from the initial reduction back towards the original pipe gradient.

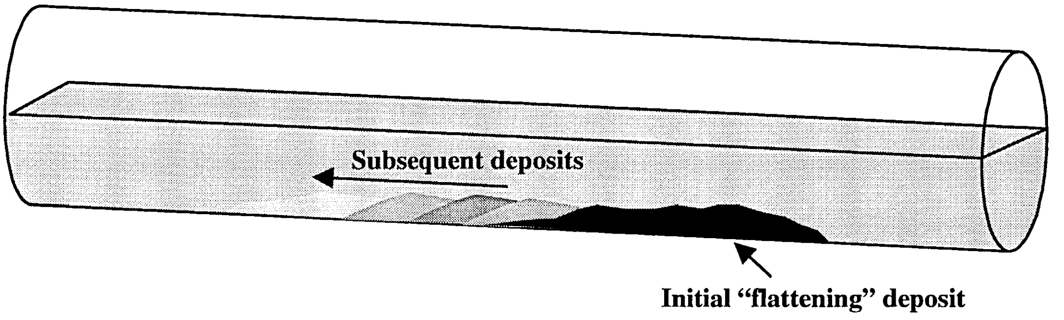


Figure 4.69 – Deposition pattern

3. The bed will continue to build, but flows exiting from the area of the bed will have a tendency to accelerate as they reach areas of reduced roughness and a locally increased gradient. This acceleration will create a preferential area of erosion at the downstream end of the pipe resulting in an increase in bed gradient.

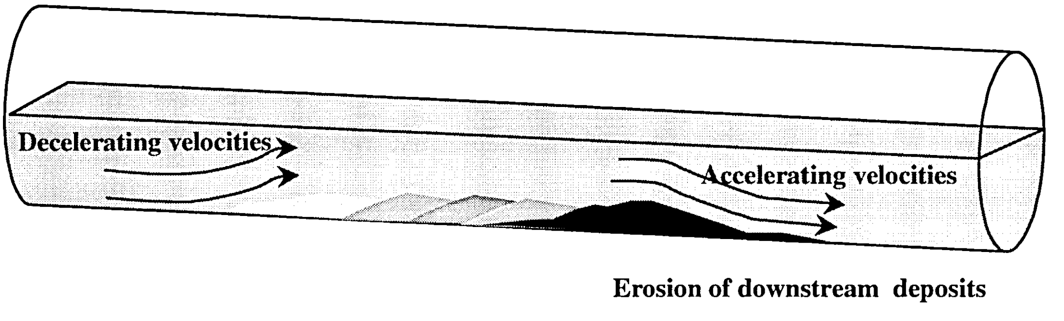


Figure 4.71 - Preferential bed erosion

Using this hypothesis and the bed level data collected by Coghlan (Coghlan, 1997), an empirical relationship was developed to represent the changes in bed gradient during the data collection period. The dependency of the bed gradient was tested against a number of variables (total sediment volume in pipe, average sediment cross sectional area, average sediment depth and rainfall characteristics). The strongest correlation was found to exist when considering the average sediment depth.

Two simple regression relationships were used to describe the dependency (Figure 4.73). Figure 4.73 shows the plot of relative bed gradient (bed gradient/pipe gradient) versus the depth of the deposited bed. Two subsets of data can be distinguished, with an apparent initial reduction in relative gradient up to a bed depth of 0.04 m. This subset is highlighted in pink. Following this initial reduction, bed depths in excess of 0.04 m are observed to generally increase the relative gradient.

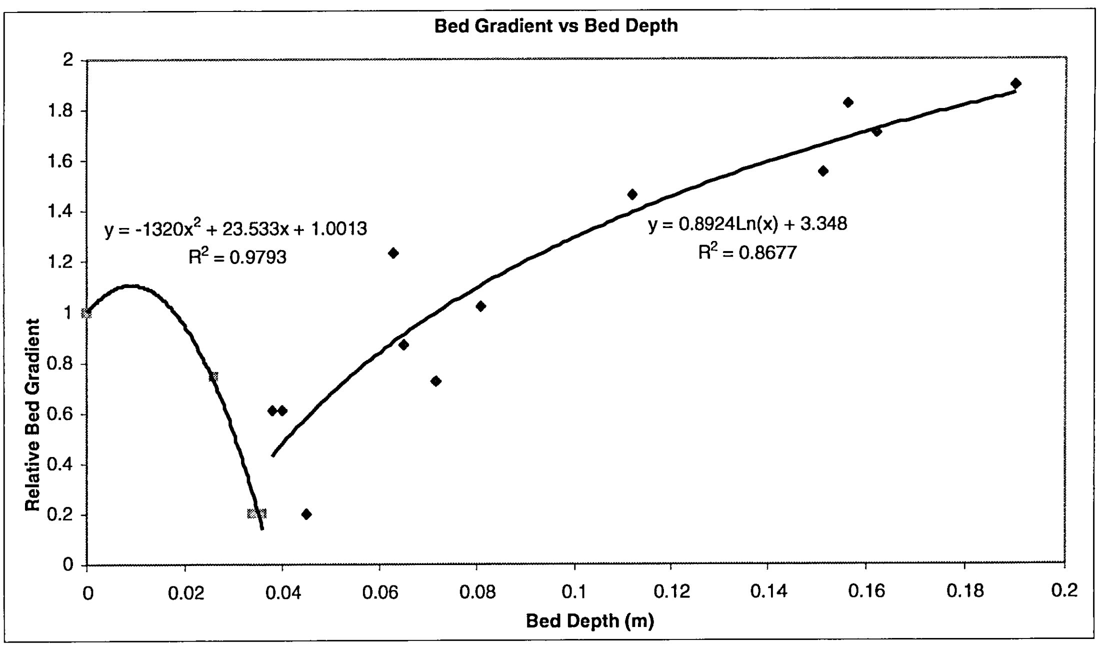


Figure 4.73 - Regression analysis of bed gradient versus bed depth

A polynomial function was used to represent initial reduction in bed gradient (Equation 4-15) following the pipe clean out, and a log function used to describe the decaying increase in bed gradient during the majority of the bed development

(Equation 4-17). It should however be appreciated that the low number of data points available for the initial reduction in bed gradient results in a misleading R^2 value of 0.98 for the data fit. Although the exact nature of the of this initial reduction remains unclear, it is evident that this initial reduction takes place.

Relative gradient = $-1320y_b^2 + 23.533y_b + 1.0013$ **Equation 4-15**

Relative gradient = $0.8924\mathrm{Ln}(y_b) + 3.348$ **Equation 4-17**

Where y_b = average bed depth (m)
Relative gradient = bed gradient / pipe gradient

Input volumes of near bed material to the model were determined initially in this case using the Arthur near bed solids model (Arthur, 1996 - Equation 2-24). This model was selected as it had previously been used successfully to model dry weather bed material in the same test pipe. A near bed solids model was chosen as it is the material moving close to the pipe invert which is most likely to settle and form deposits. Previous studies at this location (Arthur, 1996) had indicated that traditional bed transport did not occur at this location during dry weather. It is appreciated however that this relationship will not be suitable in all locations.

4.7.3 Model Results

The modified model was applied using the historic flow and sediment bed data recorded during the Coghlan surveys (Coghlan, 1997). The initial conditions used were those measured at the first survey. The model state was then altered to account for the cleaning processes, with the details used from the first survey after cleaning as initial conditions.

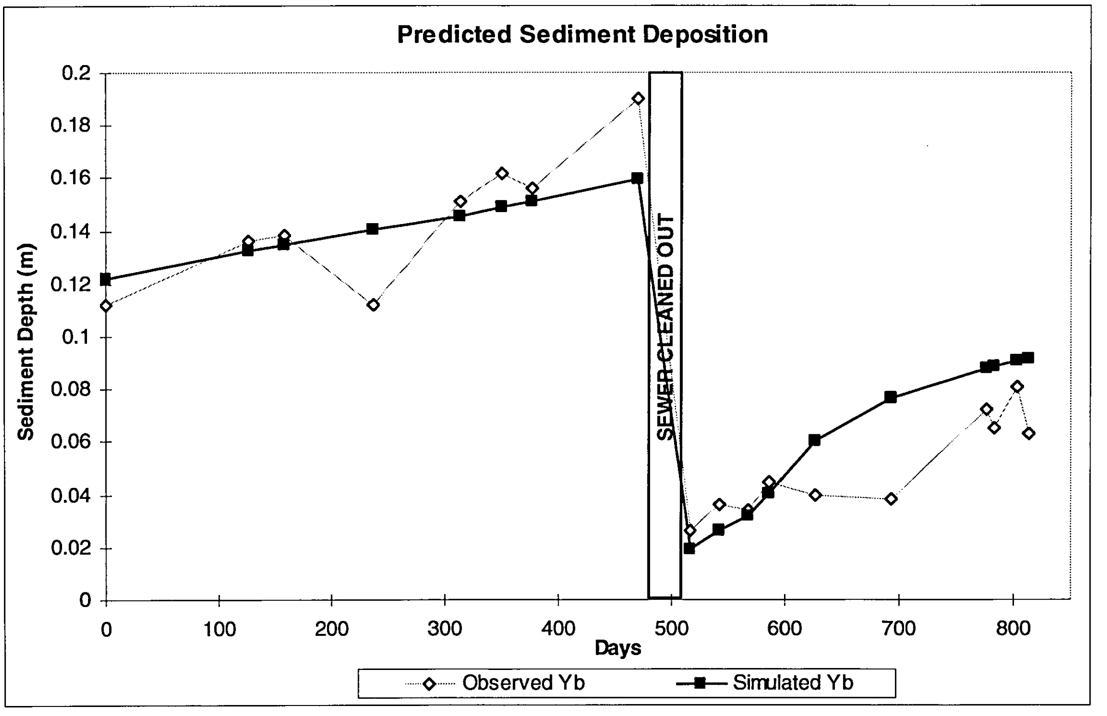


Figure 4.75 - Application of deposition quantity model

The results of this modelling exercise can be seen in Figure 4.75. In general the levels of deposition are predicted accurately, with the general rates of deposition also well represented. The initial "S" shape of the measured data following the clean out is replicated although in the modelled data set, the increase in levels occurs too early. This is believed to be due to the fact that rainfall erosion effects were not represented within this development model. The erosion component was developed separately (Section 4.8) and then later combined with the deposition model. A reduction in measured sediment levels between days 600 and 700 indicate that erosion effects have been significant at this time, thus delaying the rapid increase in sediment levels.

The model was used to identify reasons for this initial "S" curve, with the dominant factors of deposition assessed at the various stages of the curve. At the outset of the curve, the pipe is clean and resistance is minimal resulting in a slow gradual build up of sediment deposits. As the deposits increase in size (and width), their influence on the flow field becomes more pronounced, resulting in increasing deposition rates. Then as the bed develops further, the increasing bed gradient becomes more influential and increases local velocities resulting in a gradually reducing rate of sediment deposition. All of these phenomena are represented in the model either explicitly (in the case of the bed gradient) or indirectly (in the case of the bed width factor).

Figure 4.77 shows the uppermost layer of the Fraser model for the dry weather flow deposition model.

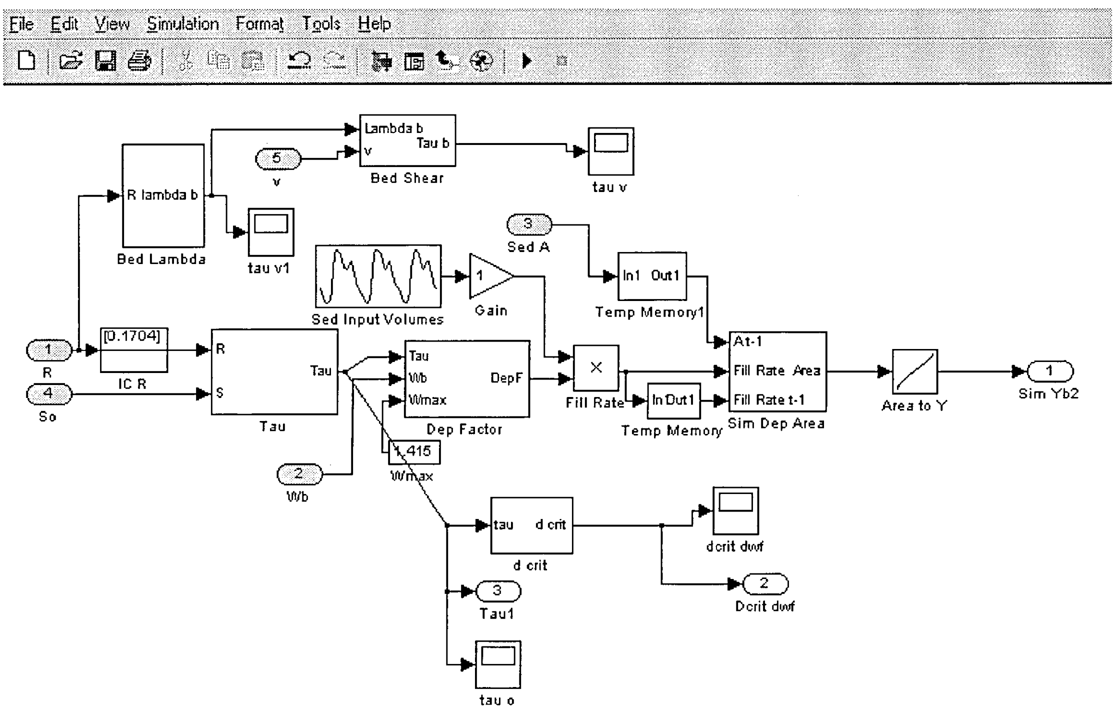
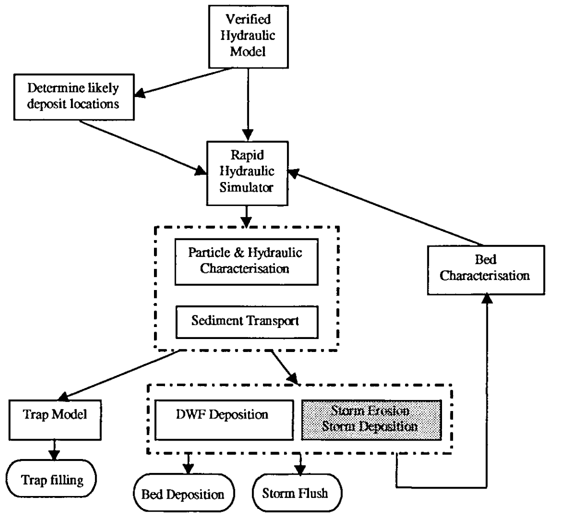


Figure 4.77 - Fraser model (uppermost layer) for dry weather deposition

The most significant limitation of this dynamic modified model is that it is unable to model erosion events. The effects of this can be clearly seen in Figure 4.75 as the observed data occasionally falls away from the modelled line at the times when erosion has taken place. It is also possible that storm deposition is responsible for some of the rapid jumps in measured data. Although this tool could be used to estimate the sediment build up in combined systems, the limitations discussed above necessitate the inclusion of a storm deposition and erosion model.

4.8 Sediment Erosion Model



It is clear from the work described above, that sediment erosion can play a significant role in the development of a deposited bed. However, few studies to date have considered this effect explicitly.

A review of methods for determining erosion rates in sewers was carried out. This yielded very little in the way of appropriate techniques. Large amounts of literature are available on the theory of granular and single sized particles (Ashley & Verbanck, 1996; Wotherspoon, 1994). However, these methods are not appropriate in areas where cohesive deposits form. In addition to this, recent research has shown that even granular sediment beds with mixed grain sizes can offer significantly more resistance to shear forces than can be determined using traditional sediment transport theory (Rushforth, 2001; Tait et al, 2003).

Skipworth developed the most recently developed sewer sediment erosion model (Skipworth, 1996). This model was assessed in an attempt to apply it to complex sewer networks rather than the individual laboratory test beds used previously. This work determined that the details of sediment data required to apply the method make it unworkable at a large scale, even within the realms of a detailed research project.

The only model tested extensively in the environment of real sewers that considered the cohesive, and consolidating strength influences was that of Wotherspoon (1994). The Wotherspoon model uses the bulk properties of a sediment deposit (density, S.G., moisture content & depth) to produce a curve of increasing shear strength with depth. The sediment is then eroded by the applied shear stress until the shear strength of the bed matches the applied stress. Further details of the Wotherspoon model are provided in Chapter 2. However, the principal variables are given by:

$$yield\_strength = \tau_y = \exp^{18.3865} m^{-3.1682}$$ **Equation 4-19**

Where m = moisture content of sediment

$$\max\_erodible\_bed\_density = \rho_e = \frac{SG\rho_w + e\rho_w}{1+e}$$ **Equation 4-21**

Where e = m SG

$$erodible\_depth = H_e = H_o - \left[H_o \left(\frac{\rho_e}{\varsigma \bar{\rho}_o} \right)^{-\frac{1}{\xi}} \right]$$ **Equation 4-23**

Where H_e = erodible depth

H_o = initial average bed depth

$\bar{\rho}_o$ = average initial bed density

ς and ξ are dimensionless coefficients

The principal limitations of the approach are that:

1. The equations have been shown to be sensitive in changes to the dimensionless coefficients ζ and ξ. (Wotherspoon, 1994) These factors control the erodible density and maximum erosion depth respectively. Wotherspoon advises that these factors be used for calibration against a measured subset of data.
2. Although originally devised for a time-based simulation, outwith an individual event, this assumption is untrue. This results from the use of the original sediment depth in each set of calculations; hence each set of calculations must be used only within each event and not continuously.
3. Although the model calculates "deposition" this is really negative erosion. Whilst this may be indicative that some deposition may take place at this time, it cannot be used to estimate deposition depths.

These limitations were addressed through the use of fixed values of ζ and ξ (previously used by Wotherspoon in the same sewer), the consideration of a new erosion simulation for each rainfall event and the removal of the "deposition" calculation. The resulting model is therefore designed to give the maximum level of erosion for a rainfall event.

The variables used within the model are therefore:

Variable	Source / Comment
Shear stress	Determined by hydraulic model
Sediment bed bulk density	Measured average (1850 kg/m^3)
ς	Assumed constant (0.68)
ξ	Assumed constant (0.347)
Sediment bed depth	From modelled data

The method was tested initially against the data collected in the original Wotherspoon study to ensure that the same performance was obtained by the modified model up to the point of maximum erosion. Following this initial testing, the model was then verified against a new event occurring in the same sewer.

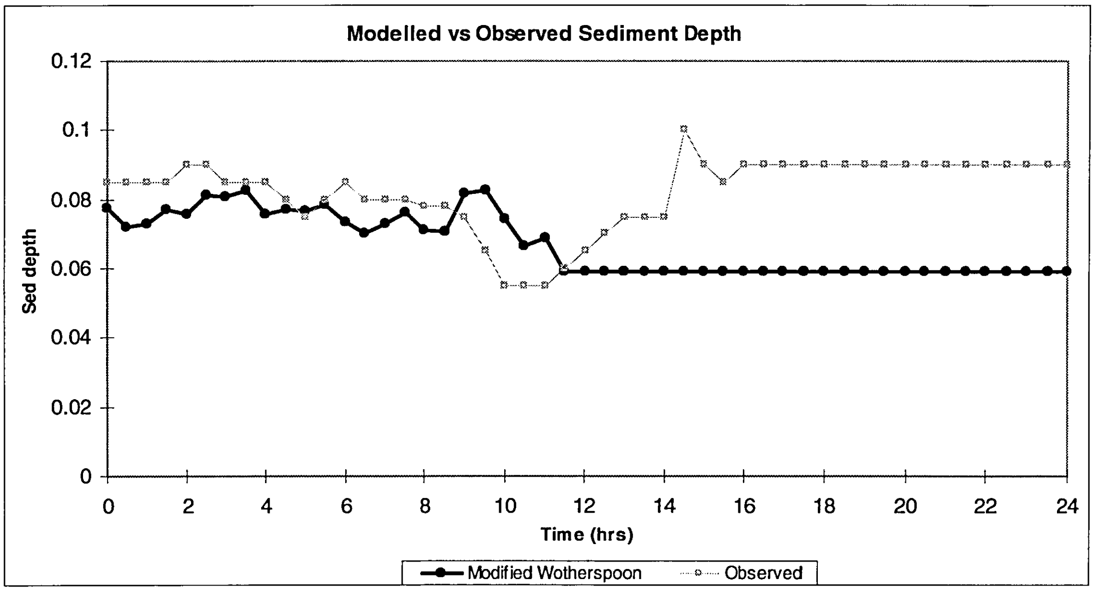


Figure 4.78 - Testing of modified Wotherspoon model

Figure 4.78 (above) shows the results from this model testing. As can be seen, once the maximum erosion depth is reached, no deposition is calculated. Throughout the model testing, the peak erosion was represented to an accuracy of between –10% to +20%. As this model was to be used in conjunction with a separate deposition model, no attempt was made to try to represent the deposition immediately following the storm.

Figure 4.80 shows the uppermost layer of the portion of the Fraser model for the erosion of the sediment bed.

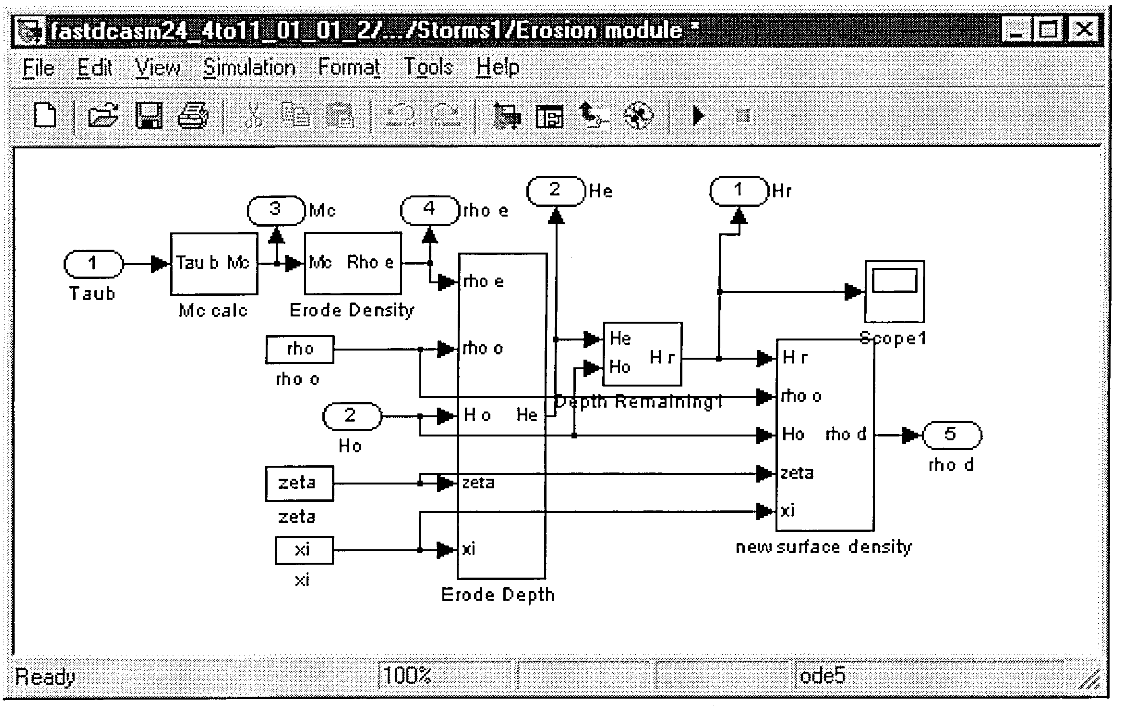
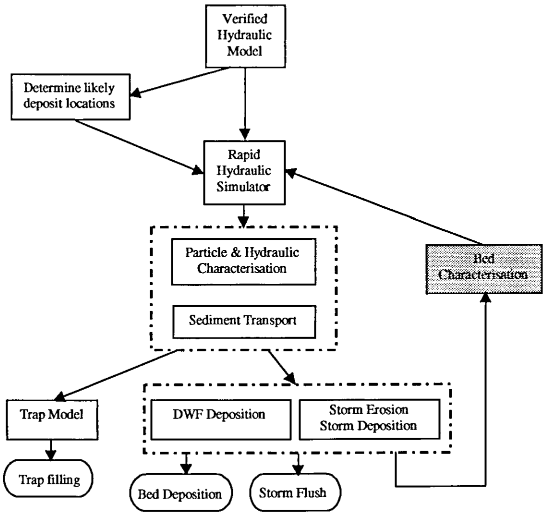


Figure 4.80 - Fraser model (uppermost layer) for sediment bed erosion

4.9 Sediment Bed Characterisation



In addition to modelling the individual processes associated with sediment movement, it is important that their interdependency and influence is also represented. The most significant of these interdependencies is the loop of influence between the hydraulic conditions (and therefore the rate of sediment deposition) and changes in pipe geometry as a result of sediment deposits. The influences are threefold:

1. The reduction of available cross-sectional area;
2. Increased pipe roughness;
3. Changes in sediment bed gradient.

A review of available data on the long-term evolution of sediment deposits revealed a general dearth of applicable knowledge. However, the model is able to provide volumes of deposition that are then converted into depths of sediment using a prior knowledge of the pipe's geometry. This information can therefore be used to define the updated pipe and bed cross-section at each timestep.

Increased pipe roughness was accounted for through the calculation of a composite roughness for a given cross section using default values for pipe and bed roughness according to pipe conditions. It is recommended that CIRIA report 141 (Ackers et al., 1996) recommendations are used to define these defaults.

As an alternative method of defining bed roughness, a routine was also implemented that allowed the knowledge of the deposited particle sizes to be used to determine the maximum roughness of the sediment bed using the formula:

$$k_b = 2.41 D^{0.61} d_{50}^{0.39}$$ **Equation 4-25**

where: k_b = bed roughness (mm)

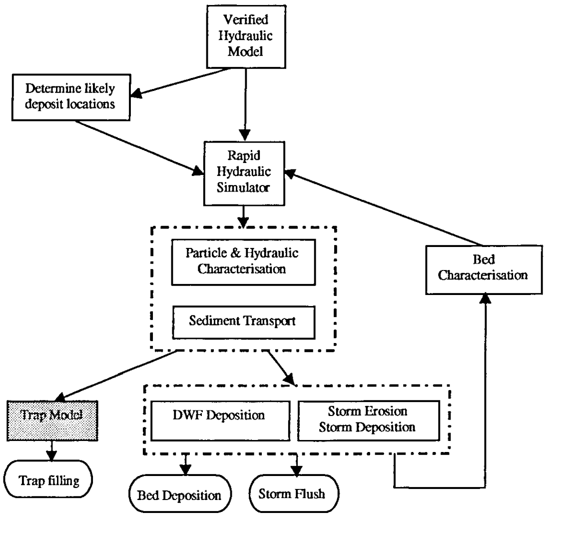
D = pipe diameter (mm)

d_{50} = mean particle diameter (mm)

However, as this relationship was developed to provide a maximum bed roughness and not an average, this approach was not used in the model testing.

Changes in bed gradient were represented using a long-term data set from the Dundee Murraygate sewer. This data set was used to determine a correspondence between the depth of deposit and the gradient of the sediment bed during its development. Further details of this are provided in Section 4.7.2.

4.10 Sediment Trap Model



The aim of this study is the provision of methods suitable to assist sewerage practitioners in the management of sewer solids. In order to effectively manage drainage sediments, a strategy must allow not only for the prediction of sediment behaviour in a system, but also offer the modelling of a sediment control method. The method of control investigated during this programme of research is that of sediment traps.

Two types of trap were considered:

- Open trap configuration – where an online sump in the sewer collects solids over the entire length of the trap;
- Partially covered or slotted configuration – where the top surface of the trap is partially covered, leaving a central slot through which sediments can deposit.

4.10.1 Initial Trap Model

The literature detailed in Chapter 2 highlights the lack of guidance currently available for sediment trap design. Trap design is a complex problem as, despite advances from earlier understanding (e.g. Ashley et al, 1992; Ashley et al, 1995; Dartus & Alquier, 1985; Bertrand-Krajewski et al., 1996), knowledge is still developing to determine where sediments will be a problem in a system, or what the nature of those sediments are likely to be. It is clear that the predominant 'near-bed' solids will vary both temporally and spatially in a system. At the time of the development of the initial trap model, the self selecting sediment transport model (Section 4.6) was not available. Consequently, for the initial test, two of the most recently developed relationships were used. These were selected on the assumption that:

- During dry weather the solids in transport in steep trunk sewers will comprise a small particle, granular bed load, with finer more organic material transported in flatter sewers.
- During Wet weather any fine, organic particles previously moving near the bed are brought into a suspended phase and are replaced by dense, granular material (Jefferies and Ashley, 1994, Arthur, 1996).

A prototype trap fill rate model using two modern "near bed solids" (NBS) transport equations was developed and applied to the Meadowside Silt Trap situated on a length of trunk sewer in the city centre area of Dundee (Chapter 2).

The model assumptions made at the outset of the investigation were:

- The trap collects and retains near bed material represented in 2 ways:
  - during dry weather: organic / inorganic mixed solids;
  - during storm flows: mainly inorganic, granular material.
- The trap captures and retains all bed material arriving at the trap inlet.
- Flow characteristics for the site are accurately represented using the HydroWorks simulation model for Dundee City Centre.

- Sediment characteristics, where not measured, can be accurately estimated using alternative Dundee data.

At this early stage of the analysis, an initial review of potential relationships for sediment inputs was carried out. It was reasoned that under dry weather, an approach based upon field data considering total solids best described the material moving near the bed. However, during storm flows, a large proportion of the organic content of the near bed material is re-entrained, thus leaving predominantly larger inorganics at the near bed region. The data collected in this study support the assertion that the material transported near the bed during storm events are principally granular. Sediment samples taken from full size sediment traps reveal granular layers coincident in time with wet weather periods. The relationship of Arthur (Equation 2-24) was selected to represent dry weather flow inputs, as the Meadowside site had been used to test the applicability of Arthur's relationship previously (Arthur, 1996). As a result of it's recent development and the fact that it was one of few relationships to be developed particularly for granular sediments in pipes, the relationship of Perrusquia & Nalluri (Equation 2-19) was used to represent wet weather solids transport.

No attempt was made to account for the 'settlement' of suspended solids in the trap, sediment washout or consolidation for the initial model as at the time of initial development, these phenomena could not be accurately quantified under the data collection programme. These effects were also considered to be minor compared to the influence of the near-bed material and bedload.

Incoming flow characteristics (used as inputs to the sediment models) were derived from a fully calibrated HydroWorks model. Rainfall data over the period of trap measurement were used and ranked in an order of significance. The events were then simulated in order of total rainfall depth until the size of the events was no longer sufficient to raise flows above peak dry weather flow for the site (recorded using flow monitors).

The cumulative trap fill rates were then determined through applying the appropriate sediment transport relationship to the appropriate flow conditions. Dry weather inputs were averaged at hourly time-steps, with the storm inputs calculated over 2-minute time-steps.

The outputs for this initial trap model were then compared to measurements of trap filling that were measured over the same period. As a result of access problems for the site, few measurements of sediment volume were taken during the early stages of trap filling.

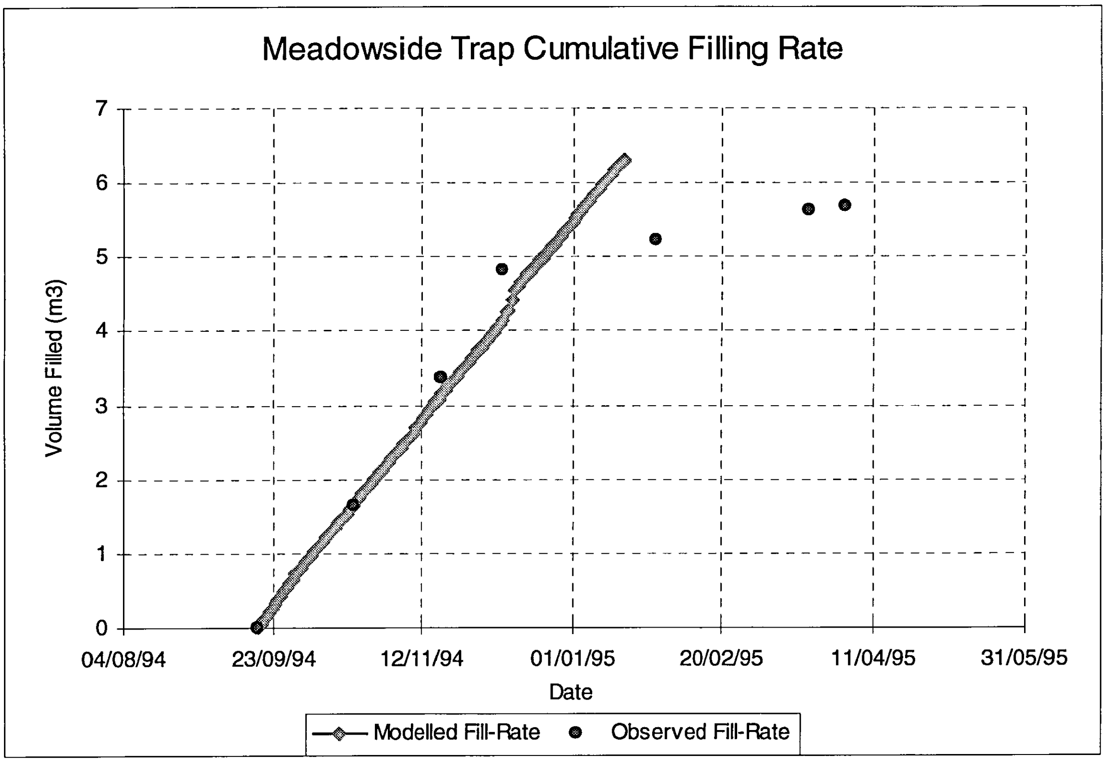


Figure 4.81 - Initial trap fill model results

Figure 4.81 (above) shows the results of this model over the test period. Initial observed fill rates are closely matched by the model, with a general underestimate apparent after approximately 30 days. A close inspection of the modelled data over this period indicates that dry weather flow solids are dominant in the pattern of fill rate, as overall, fluctuations caused by rainfall effects are generally small (Figure 4.83). Although the resolution of the sampling times does not allow this assertion to be supported by the observed data, sediment samples taken from the trap during the initial filling period showed the characteristics of the trapped particles to be broadly similar to those of the material in dry weather transport (Vollertsen et al., 1998). This dominance of dry weather flow solids cannot however be assumed throughout the entire filling period of the trap.

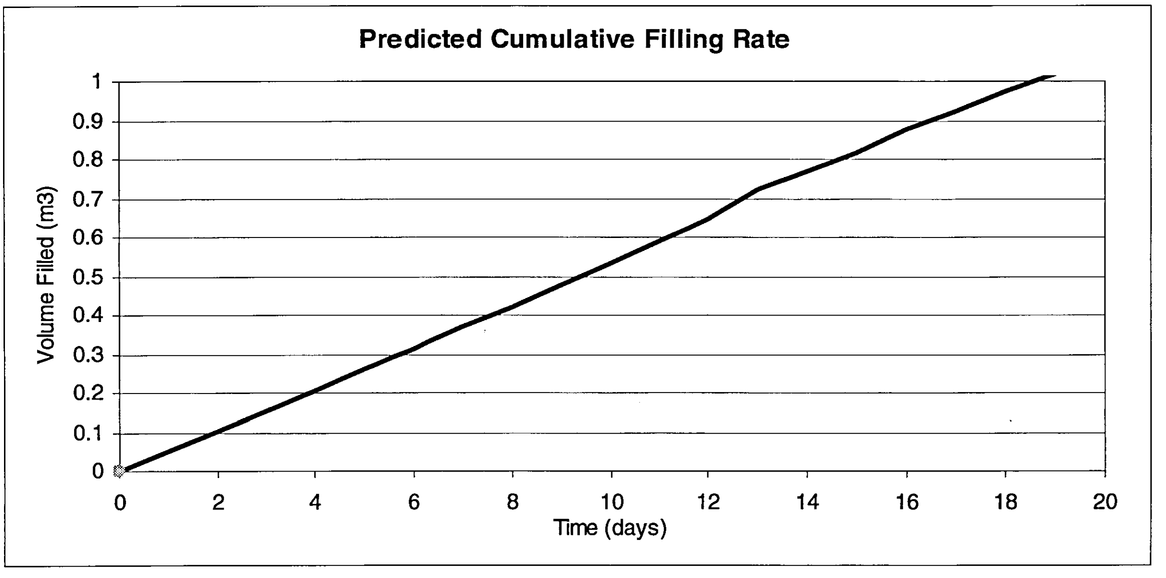


Figure 4.83 - Predicted fill pattern (days 1-20)

Between approximately 60 and 80 days, the observed data show an increase in the overall rate of sediment deposition within the trap. Only a minor increase in the rate of sediment deposition is calculated by the model over this time. This period was characterised by increased rainfall, suggesting that the storm model is not replicating the full extent of storm impacts.

Following this increase in trapping rate, the observed data flattens off as the trap nears its filling capacity. These effects were never included in the initial model as no previous data existed with which to assess this effect. Consequently the data sets diverge significantly. The principal reasons for the reduction in trapping efficiency after approximately 80 days are the alteration of the trap geometries and consequently of the detailed hydraulics of the structure.

Initial investigations of the effects of this were carried out using the Computational Fluids Dynamics (CFD) package Fluent by Buxton (Buxton, 2003). These experimental simulations indicated that as the trap fills, the strength of the re-circulation patterns within the trap gradually reduce with the general flow direction within the trap slowly becoming more elongated and flow direction dominated. In addition to this, the overall depth available for the settlement of solids becomes less, resulting in only particles with higher settling velocities being trapped. These effects are shown diagrammatically in Figure 4.85 and Figure 4.87.

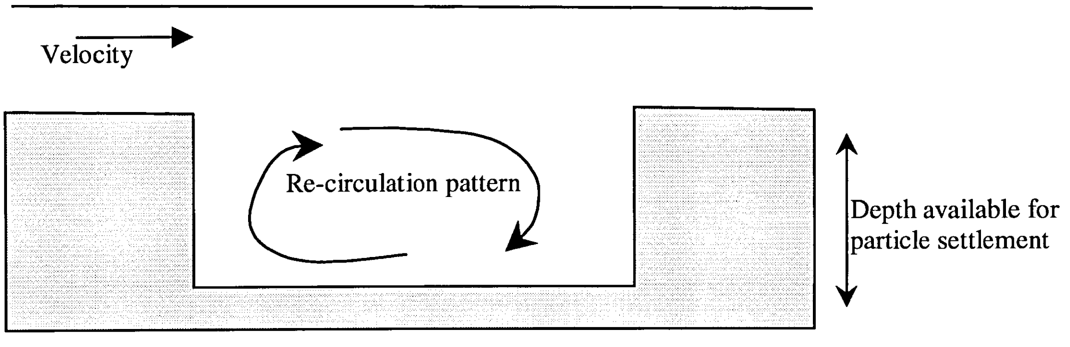


Figure 4.85 - Empty trap re-circulation patterns

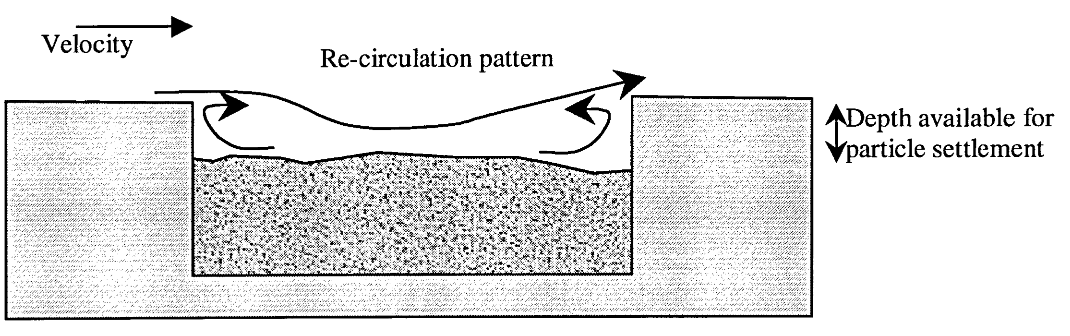


Figure 4.87 - Partially filled trap re-circulation patterns

The initial trap fill modelled results of this study, although clearly restricted by the use of some estimated parameters, provide a level of accuracy which warranted further investigation and data collection. The effect of each of the parameters being used in the equations was evaluated via a series of sensitivity tests. The results were found to be most sensitive to the estimated near-bed velocity. A greater level of sensitivity was expected to be found for the rainfall parameters (peak intensity, total depth and duration). It is suggested that the sensitivity of any particular trap to rainfall parameters is site specific, as within this study, the nature of the trapped sediment types (predominately either storm or dry weather flow) was observed to vary significantly, with trap hydraulics. The test trap used in this particular case (Meadowside, Dundee) was found to trap a significant portion of dry weather flow material resulting in a low overall sensitivity to storm flows during the early stages of trap filling.

4.10.2 Advanced Trap Model

The principal limitations of the initial trap model were discussed in Section 4.10.1. These can be summarised as:

1. Efficiency of trap not adjusted as trap fills.
2. Efficiency of trap not related to either incoming particle size or trap hydraulics.

At the outset of the study, it was anticipated that these modelling limitations could be addressed through the use of the outputs from a detailed CFD study into the behaviour of trap hydraulics and sediment behaviour. This CFD study was run in parallel to the investigations described within this thesis and used a combination of CFD and laboratory analysis (Buxton, 2003). The principal findings of this work are summarised in Section 2.6.2.2. However, only limited outputs were found to be applicable to sewer sites.

4.10.2.1 Advanced Trap Model Development

As a consequence of the difficulties described in Section 2.6.2.2, an alternative model was developed as part of this study using a combination of the experiences and insights gained during the CFD study, and traditional sediment transport analysis.

For the purposes of this trap model, it was initially assumed that the efficiency of the sediment trap is based on:

1. The initial trap geometry (when clean);
2. Changes in trap geometry during trap filling;
3. Incoming particle size and density;
4. Incoming hydraulics (and hence sediment transport mode).

It was also assumed from the outset of model development that the processes of trap filling are different for bed and suspended load. The evidence for this comes from the CFD and laboratory studies undertaken in parallel to this investigation, and also field observations made during the data collection phase. The evidence of both of these studies pointed towards a gravity dominated process for bed load and a turbulent or flow pattern dominated process for suspended material. The sediment trap model must therefore represent both of these processes and sum the resulting rates of deposition to predict the rate of trap filling. This process is shown diagrammatically below in Figure 4.89 and the required models are described further in the following sections.

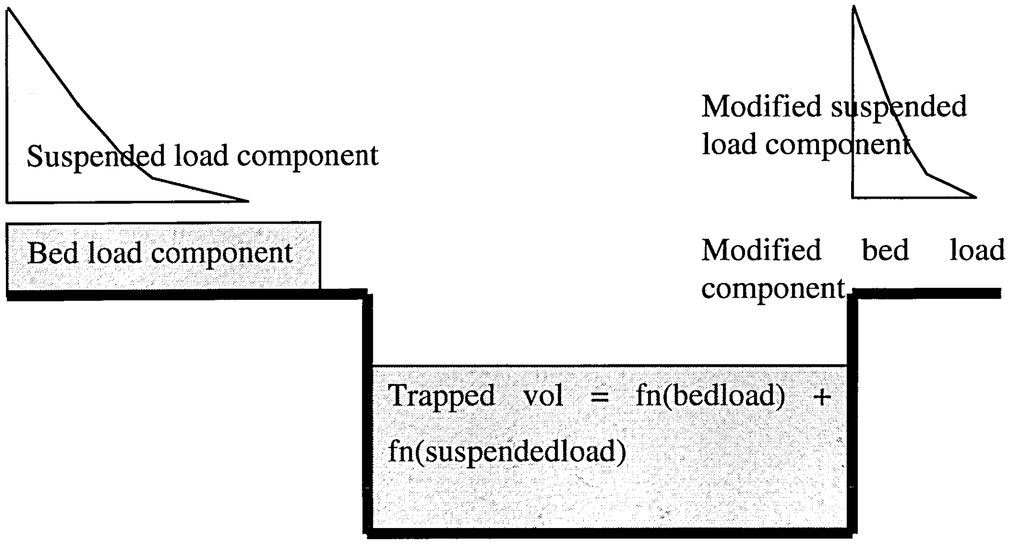


Figure 4.89 - Components of trap filling models

4.10.2.1.1 Bed-Load Trapping Model

It is clear that material moving as bed-load is more dominated by gravitational effects than suspended material whose motion is linked more strongly with turbulent and hydraulic properties. It was therefore assumed that bed-load material entering the trap settles and is retained unless the filling effects of the trap dictate otherwise (e.g. trap full). The rate at which bed-load material enters the trap is based upon the length over which settlement can occur (trap length or slot width) and the capacity of any particle to pass over that length (i.e. particle jump length).

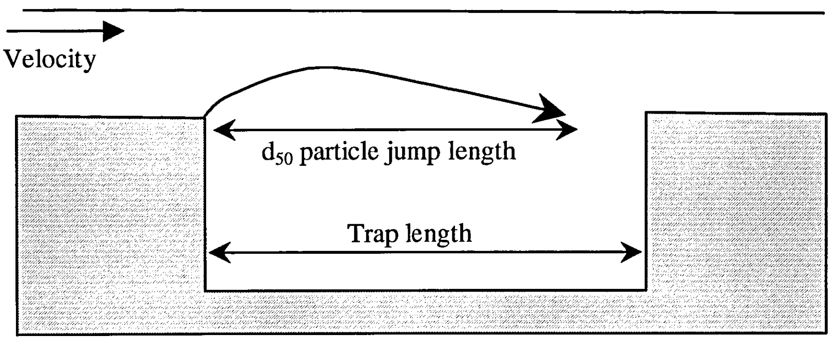


Figure 4.90 - Conceptualisation of bed-load trap model

Although only a single particle size is used to represent the bed-load material, the procedure adopted attempts to account for a range of particle sizes by using an assumed linear particle size distribution over the range of maximum and minimum sized bed-load particles. In this way a very simple retention efficiency model could be developed using only the particle jump length and trap or slot size.

Within this model, the transport mode present at the trap inlet is determined using either Ackers-White dimensionless grain number or the particle's sedimentation parameter. If pure bed load is present, the maximum transportable particle diameter is determined using Shields' criterion (Shields, 1936), and the minimum size of bedload particle determined using the sedimentation parameter (assuming granular material). These sizes are then compared against incoming particle characteristics. The jump lengths of the maximum and minimum sized particles are then calculated using the procedure first set out by van Rijn (1984) and later used by Arthur (Arthur, 1996). The equations used in this method are shown below:

$$J_m = 3D_*^{0.6}T^{0.9}d_{50}$$ **Equation 4-26**

Where: J_m = the mean jump length
d_{50} = median particle diameter
D_* = dimensionless particle diameter (see below)
T = shear ratio (see below)

$$D_* = d_{50}\left[\frac{(s-1)g}{\nu^2}\right]^{\frac{1}{3}}$$ **Equation 4-28**

Where: ν = kinematic velocity coefficient

$$T = \left[\frac{(\tau_o - \tau_{cr})}{\tau_{cr}}\right]$$ **Equation 4-30**

Where: τ_o = applied shear stress
τ_{cr} = critical shear stress

The performance of this approach to determining jump length was assessed through its application over a range of particle sizes and shear stresses and its comparison to data observed in other sediment transport studies (Van Rijn, 1984; Buxton, 2003). As a consequence of the difficulties in measuring jump length in sewers, only laboratory-based data were available for comparison (van Rijn, 1984). In general the equations were observed to perform within an error band of –25% to +30% (Van Rijn, 1984).

As a result of the form of the jump length equation, the relationship between jump length and shear stress varies almost linearly. However, a more complex behaviour is apparent when varying the particle size. Figure 4.92 shows how the jump length varies with particle size using an applied shear stress of 2 N/m^2. A maximum jump length can clearly be seen to exist at a diameter of approximately 1.2 mm. This behaviour was investigated further using a range of shear stresses and through expressing the particle diameter by using the sedimentation parameter, η.

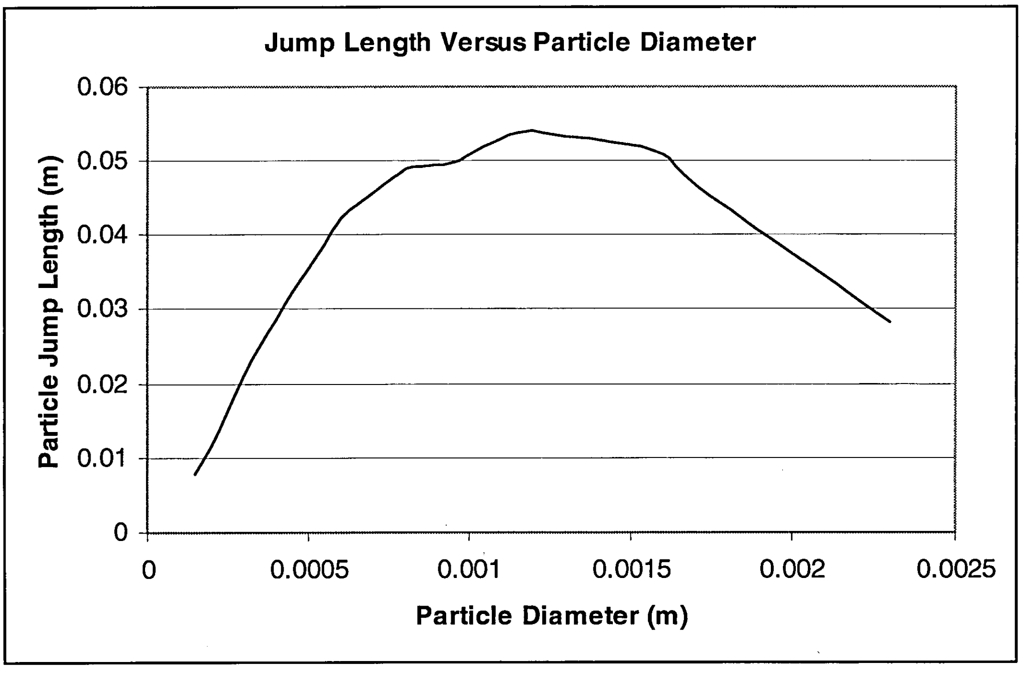


Figure 4.92 - Variation of jump length with particle diameter

It was noted during these investigations that the peak jump length seemed to frequently occur at similar levels of sedimentation parameter, regardless of the particle sizes or shear stress used.

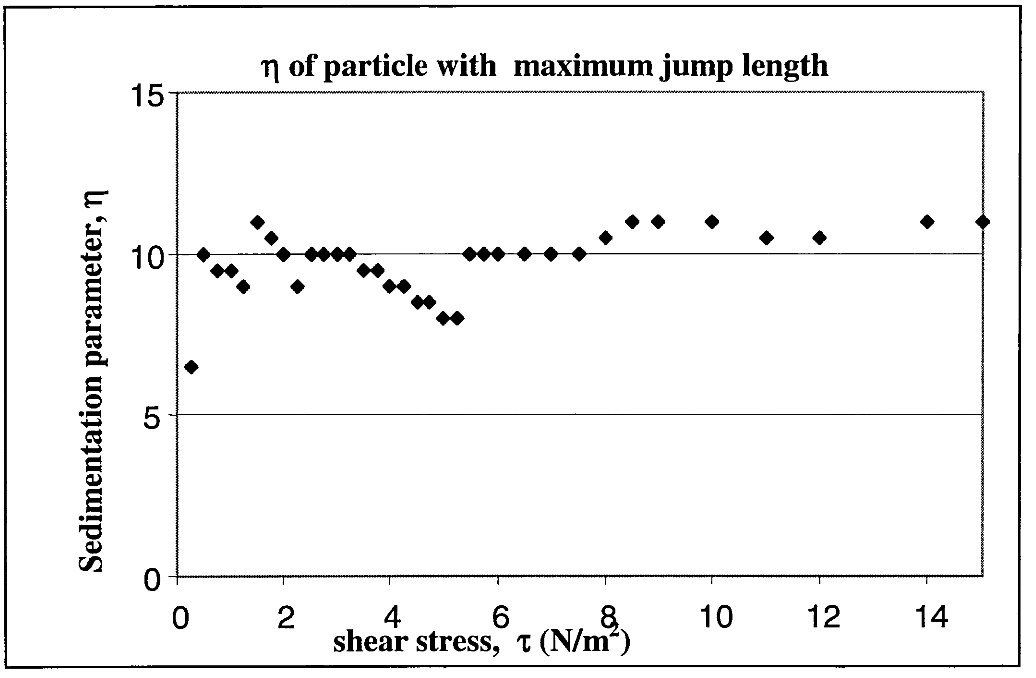


Figure 4.94 - Effect of varying particle size and shear stress on max jump length sedimentation parameter (η)

Figure 4.94 shows the results of this testing. For varying particle sizes of between 0.15 mm and 5 mm, the applied shear stress was incrementally increased up to 15 N/m^2. At each stage, the sedimentation parameter at which the maximum jump length occurred was noted. As can be seen in Figure 4.94, the maximum jump length was observed to occur at values of sedimentation parameter of approximately 10. This has significant implications on the trap model, as the maximum jump length should be known in order to assess the effectiveness of any trap length. Consequently, within the trapping model, for any given shear stress, a sedimentation parameter of 10 is used to characterise the particles with the maximum jump length.

As a linear jump length distribution is assumed, a linear retention relationship results. When the trap length is greater or equal to the maximum bedload jump length, a retention efficiency of 100% is assumed as no particle in bed-load can pass the trap. For trap lengths of less than the maximum jump length, it is assumed that the degree of interception is linearly proportional to the ratio of trap length to maximum jump length. Hence for a trap length of 0.5 times the maximum jump length, 50% of particles are assumed to be intercepted and retained. These assumptions are intentionally crude, as no events where the jump length exceeded the trap length were recorded during the study. This is a result of the general use of long open trap configurations, and the lack of bed load transport at the test site for a partial cover configuration.

These basic assumptions result in the following relationship for the bed-load retention efficiency, e_b:

$$e_b = \frac{TL}{J_{max}}$$ **Equation 4-32**

Where: TL = Trap length (m)
J_{max} = Maximum jump length (m)
$e_b \leq 1.0$

Initial experimentation with these assumptions has shown that for the levels of shear stress generally found in sewers, a bed-load retention efficiency of 100% results. However, it should be noted that as shear forces increase, the quantity and type of material in the bedload will change. Bed transport quantity will peak at intermediate levels of shear stress (depending on particle characteristics present), and then reduce at high shear stresses as more of the material is brought into suspension. This behaviour can be replicated by using a sedimentation parameter of 10. During peak flows, the particle size corresponding to a sedimentation parameter of 10 will increase, resulting in the assumption of a coarser fraction. Within this study, the impact of these changes on bed load transport rates were only represented through the use of the increased particle size. The availability of these particles to the transport process was not addressed.

However, the general finding of 100% efficiency under "typical" conditions is in agreement with the findings of the model initially applied (Section 4.10.1), but does not however correspond with parallel laboratory studies undertaken in Sheffield (Section 2.6.2.2). These studies showed a liner reduction in trap retention with increased discharge although a calculation of maximum jump length indicates values smaller than the trap length.

An analysis of the sedimentation parameters used in the Sheffield laboratory study shows the sedimentation parameter of the particles to be in the range of between 2.9 and 6.7, with the majority of tests carried out at sedimentation parameters of less than 4. In this range, the behaviour of the particles is known to deviate from that of pure bed-load and some suspended effects become apparent. This conclusion is supported by the observations of Ali (Ali, 2002) in laboratory tests of trapping bed-load particles. As the model proposed here calculates only the material in bed-load, these effects should not be replicated.

No laboratory data were available to test the approach at jump lengths in the order of the trap length, as the applied shear stresses in the Sheffield study did not reach this level.

4.10.2.1.2 Suspended-Load Trapping Model

The trapping behaviour of the suspended material is more difficult to describe in simple physical terms, as detailed knowledge of the hydraulic flow patterns, turbulence, re-circulations and pressures is required. The three potential flow paths for a suspended particle are:

1. Suspended material passes directly over the trapping structure and is kept in the upper regions of the principal flow direction by turbulent forces.
2. Re-circulation and pressure forces pull suspended particles into the trapping structure. These particles are then carried out of the sediment trap by the same re-circulation and pressure forces.
3. Re-circulation and pressure forces pull suspended particles into the trapping structure. The same re-circulation and pressure forces are then insufficient to carry the initially suspended particle out of the trap, resulting in its retention.

Due to the complexity of the processes involved, it was decided to apply the experience of CFD and laboratory studies through the use of generic retention diagrams using varying particle characteristics, flow conditions and trap configurations. Buxton (2003) tested the effect of a number of factors on the retention performance of traps. One of the most widely applicable of the retention diagrams produced was that of retention efficiency for a range of sedimentation parameters (Figure 4.96). The plot shows retention curves produced for 3 different trap configurations (90 mm slot, 45 mm slot and 22.5 mm slot) and with both styrocell and sand particles. It is proposed that these trap configurations could be normalised (slot length/trap length) and the retention curves applied to a wide range of actual trap sizes.

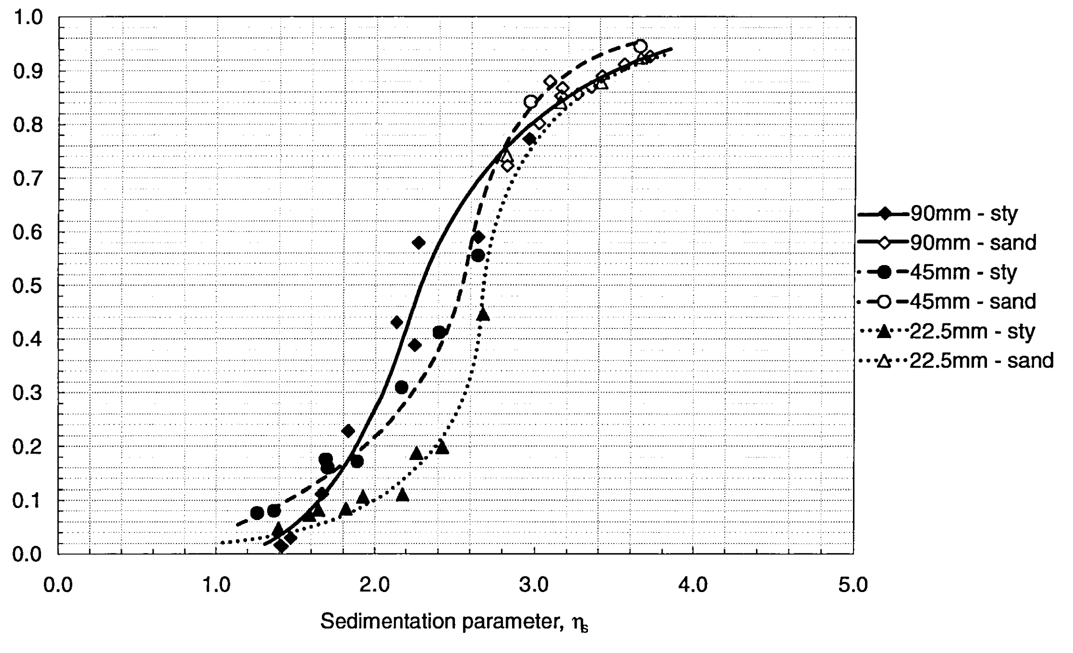


Figure 4.96 - Buxton (2002) laboratory test results

In the case of an open trap, the retention curve approximates to a solid line generated from a slot width of 90% of the overall trap length. Essentially these curves are produced from a single sized sediment particle (styrocell) used to represent suspended material. As a result of this and the complexity of determining the variable characteristics of the suspended material, it is suggested at this stage of the study to restrict the assumed characteristics of the suspended particles to those of the artificial styrocell. A summary of the characteristics of the styrocell particles is given in Table 4.22.

Sediment	Density (kg/m^3)	D_{min} (mm)	D_{max} (mm)	D_{50} (mm)	$w_{s\,min}$ (mm/s)	$w_{s\,max}$ (mm/s)	$w_{s\,50}$ (mm/s)	Range of η_s
Styrocell	1.038	0.85	1.4	0.975	6.6	17	12.3	1.16 – 2.78

Table 4.22 - Styrocell characteristics

For any given hydraulic condition, the sedimentation parameter is calculated for the characteristic particle and used to derive the appropriate sediment retention factor for suspended material. This allows the total percentage of suspended material that may be retained by the trapping structure to be calculated.

4.10.2.1.3 Trapping of Near Bed Solids Material

The inclusion of near bed solids into the analysis creates a paradox, as although the appearance of this material is similar to that of bed-load (in terms of location) much of its behaviour is in fact more in line with that of suspended material. This has a great significance when selecting which approach should be used to determine the retention of the near-bed solids. At this stage in the development of near bed solids knowledge it is proposed that the material should be treated as a suspension. This assumption agrees with the observations of Verbanck (Verbanck, 2000). Consequently the approach described for determining the retention of suspended material should also be applied to the trapping of near bed solids.

4.10.2.1.4 Overall Trap Effectiveness

As the trap continues to fill, the overall effectiveness of the trap reduces. The pattern of reduced retention is principally dependent on the trap configuration used. As no indication of these effects could be provided using the laboratory or CFD studies, a purely empirical relationship for varying retention efficiency has been developed using data collected from field studies (Chapter 3). It should however, be noted that these data pertain to open trap configurations only.

For each site, the trap filling data were averaged, and at each time step were divided by the average dry weather measured bed-load arrival rate (measured using box traps). This produces the dimensionless effectiveness of each trap throughout its period of filling. It should be noted that this method of expressing the efficiency of each trap does not take into account individual rainfall impacts as the volume of material deposited in each rainfall event could not be assessed. Consequently an assessment of any variation of trap performance during rainfall could not be carried out. This is apparent in Figure 4.98 to Figure 4.102 as effectivenesses of greater than 1 were observed in weeks characterised by storm conditions.

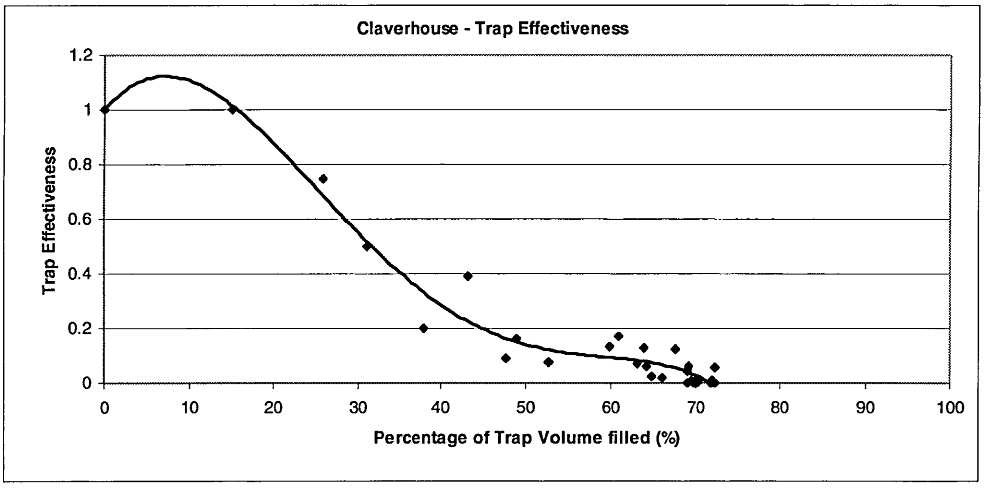


Figure 4.98 – Baldovan Road - Claverhouse average trap effectiveness throughout trap filling

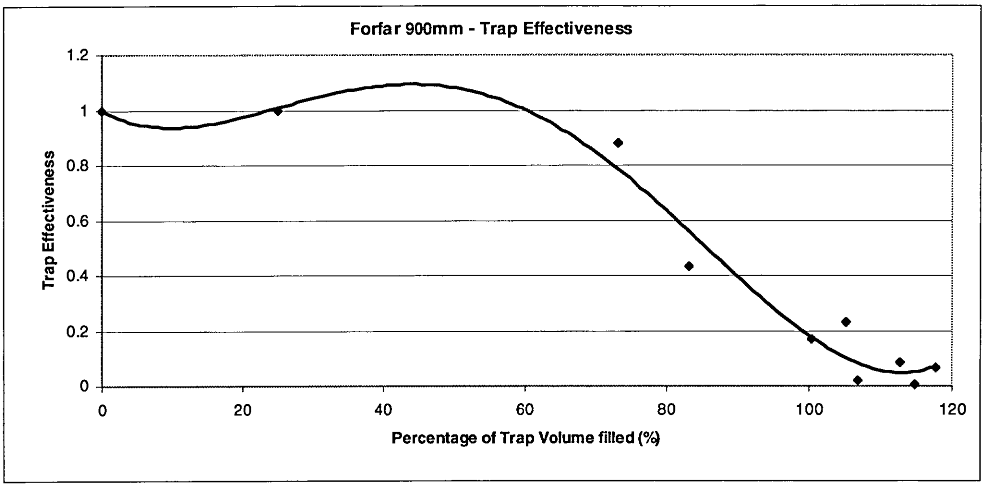


Figure 4.100 – Forfar 900mm average trap effectiveness throughout trap filling (open trap configuration)

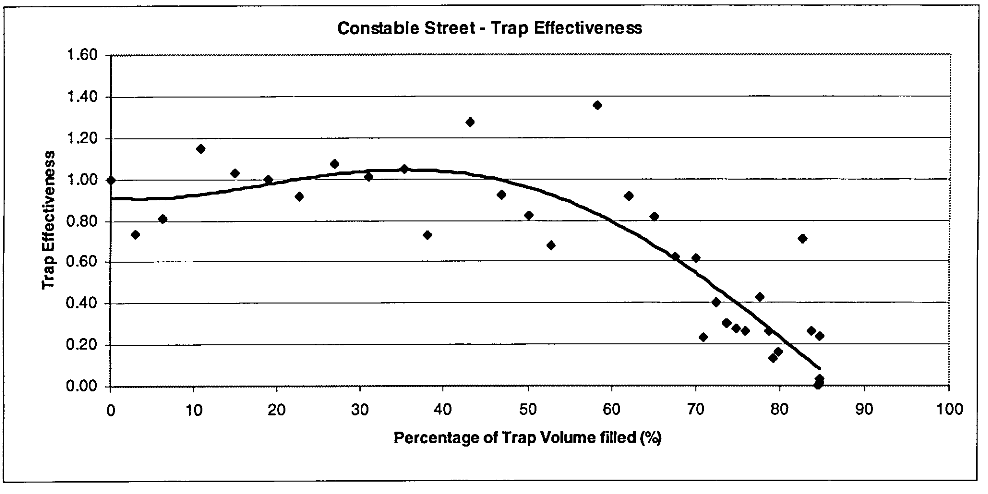


Figure 4.102 – Constable Street average trap effectiveness throughout trap filling

It was hoped that a generic effectiveness ratio could be used for all open traps. Clearly each of the curves shown in Figure 4.98 to Figure 4.102 exhibit their own characteristics. It is believed that the controlling factors in the shape of these curves are:

- Trap depth – A trap with a larger depth will take significantly longer for the effects of trap filling to impact on its retention efficiency.

- Pipe hydraulic regime & sediment loading – In the case of the Forfar trap, general conclusions are harder to make as a result of fewer data points. However, the ambient hydraulic regime and high solids loading results in high fill rates for much of trap filling. This behaviour is shown in its extreme in the final stages of trap filling where even though the full trap volume is filled, deposition continues (i.e. bed deposits continue to fill the above chamber area).

As each of these curves exhibit such varying characteristics, it is proposed that each curve should be used specifically for the trap in question. Bed-load and suspended retention efficiencies should therefore be multiplied by this overall factor prior to being used to determine retention volumes.

In the case of the slotted trap configuration, as no field data were available, it was assumed that an overall trap effectiveness factor of 1.0 is valid until the trap is full.

4.10.3 Advanced Trap Model Testing

The functions for the trapping of bedload, suspended load and overall trap effectiveness can be combined to predict the total fill rate of a sediment trap. The rate of trap filling is therefore given by:

$$Q_{fill} = \left(Q_b.e_b + Q_{sus}.e_{sus}\right)e_t \qquad \textbf{Equation 4-34}$$

Where:
Q_{fill} = sediment trap fill rate
Q_b = volumetric flow of bed load material
e_b = bed load retention efficiency
Q_{sus} = volumetric flow of suspended and/or NBS material
e_{sus} = suspended load efficiency
e_t = overall trap effectiveness

This trap model was applied with no verification to each data set. No model verification was carried out as the only data sets available were those used in model development. The SIMULINK programming environment was used to generate time varying results using the measured inputs of flow and calculated sediment concentrations. The procedures developed in Section 4.6 (sediment transport model) were used to determine the transport rates.

Measured bed load and suspended load data were available in each case to allow calculated dry weather transport rates to be verified. Minor calibrations were required at this stage as a result of the variability of solids loadings between the sites (Section 3.5). In the case of the Claverhouse trap, the dry weather suspended load was omitted as the samples taken at this site showed extremely low loadings (negligible dry weather bed load and approximately 50 mg/l suspended concentrations). This is a result of the low number of foul connections upstream of the trap.

In contrast, the high solids loading experienced at the Forfar site (dry weather TSS concentrations of approximately 250 mg/l) required the assumption that the flow was at transport capacity during dry weather. This highlights the variability of potential conditions and the importance of good investigative data collection prior to deciding on the approaches that should be used to determine fill rates of any particular trap.

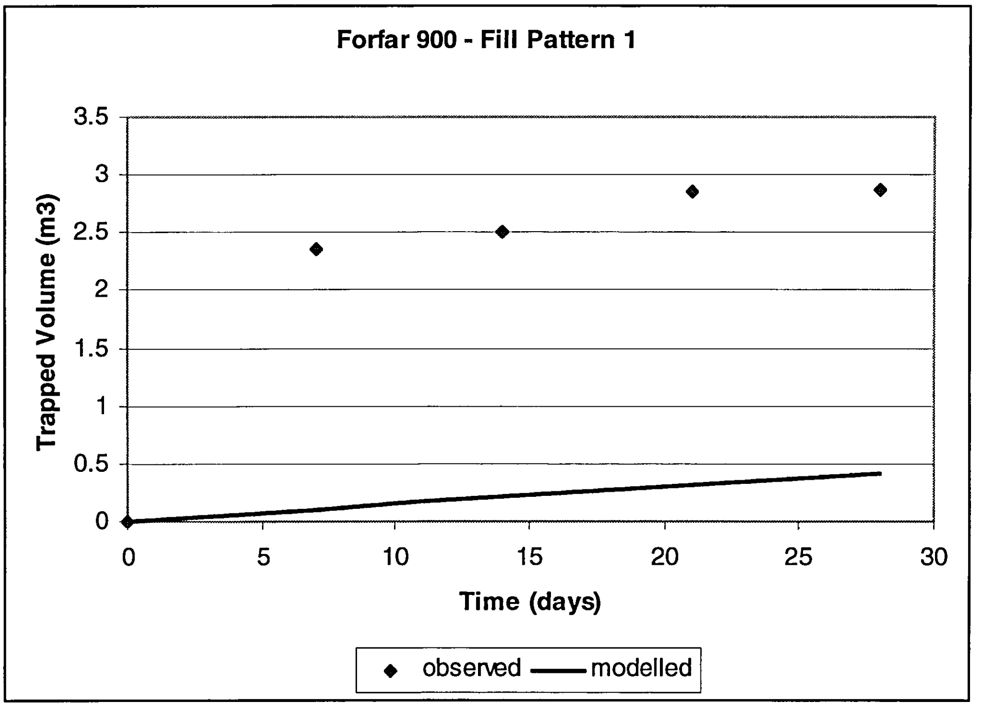


Figure 4.104 - Initial modelled Forfar trap filling pattern 1

Figure 4.104 shows the initially modelled trap volumes for the Forfar Trap. As can be seen trap fill volumes are grossly under predicted. The observed data show that sediment loadings at the trap are very large although the material is very fine. Hence initially, the solids were assumed to behave as a suspension.

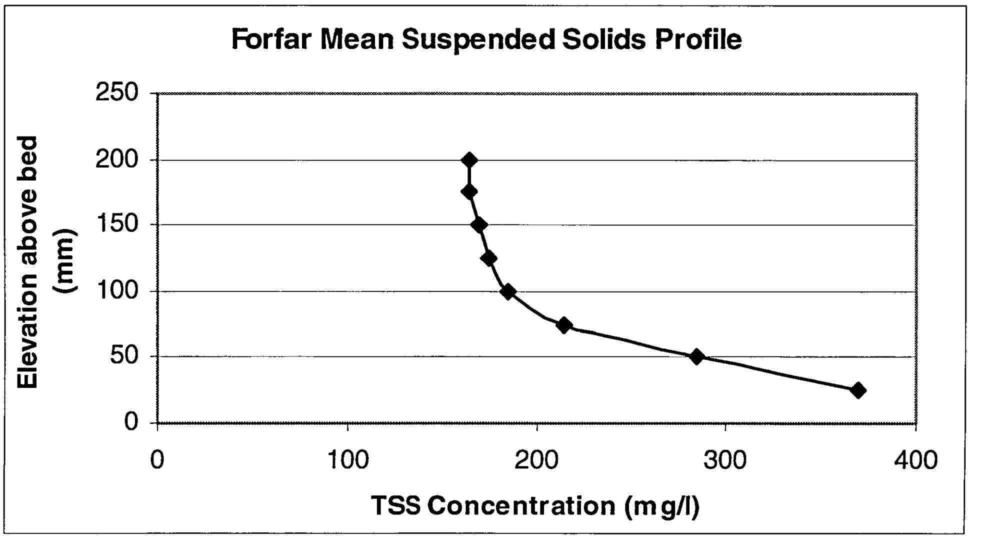


Figure 4.106 - Forfar average DWF suspended solids profile

Figure 4.106 shows the average dry weather flow suspended solids profile for the site. Under the initial assumptions of the model, and near bed solids element calculated as contributing to the trap, it was assumed to follow the retention rules of suspended material. However, it is believed that as a result of the low velocities associated with this site (<0.1 m/s) that the near bed solids material should in fact follow the rules for bed-load deposition where gravitational forces dominate.

This modified assumption was then applied to the Forfar site for two sets of trap filling data.

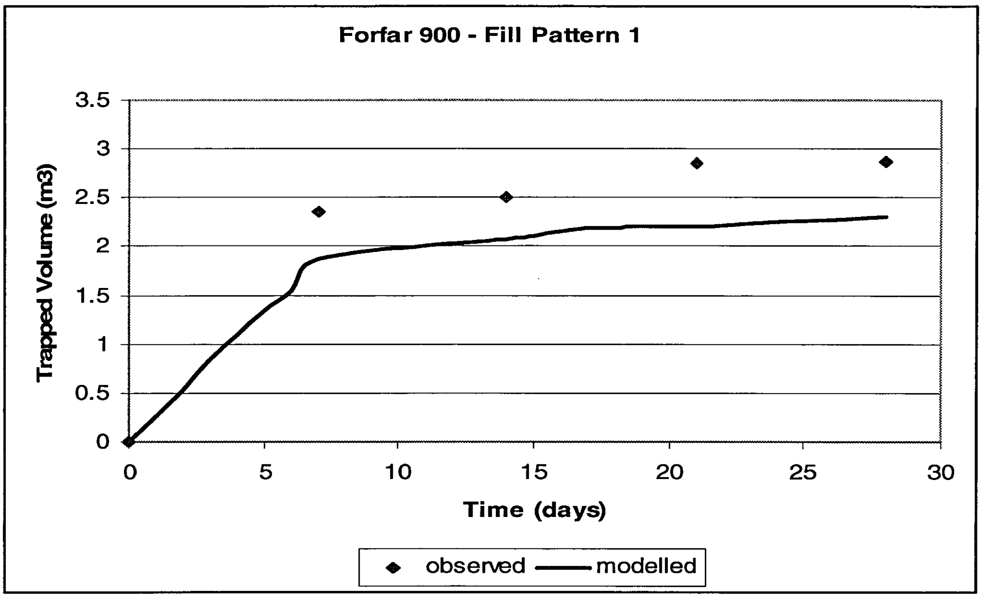


Figure 4.108 - Modified sediment transport model - Forfar trap filling pattern 1

Figure 4.108 shows the effects of changing this assumption of near bed solids behaviour. This is a clear improvement in the behaviour of the model, with a relatively constant under-prediction throughout. The general gradient between each of the observed data points is predicted accurately although the absolute values are not as precise. The assessment of the model at this site is however limited by the time resolution of the available observed data. This was a result of the very fast initial filling of the trap following cleaning.

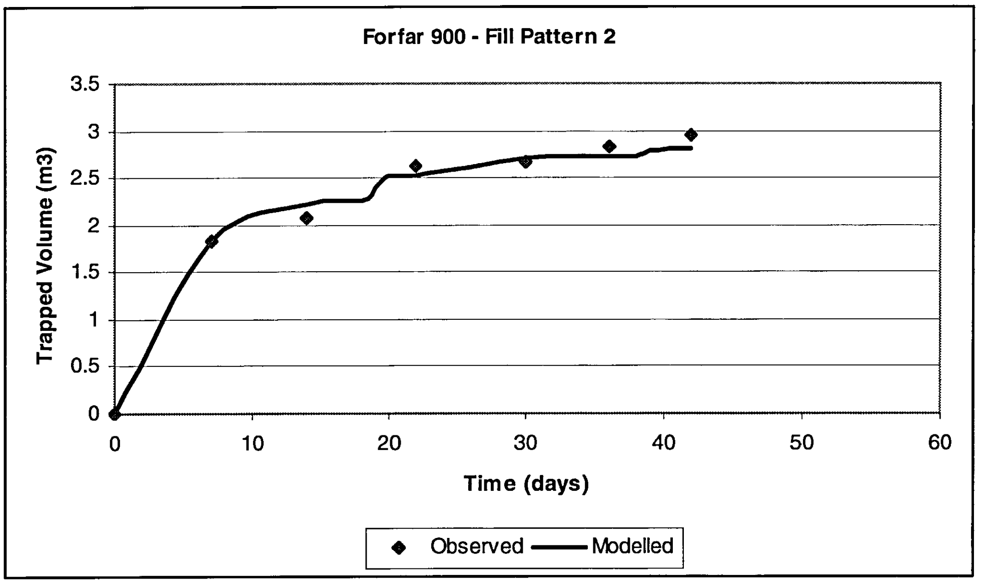


Figure 4.110 - Modified sediment transport model - Forfar trap filling pattern 2

Figure 4.120 shows the modelled results for trap filling pattern two. An improved fit can be observed for this data set. It is believed that this is a result of drier weather being experienced during data set two, as it appears that the rate of fill in both simulations is under-estimated during wet weather. It is also believed that the more constant conditions of dry weather flow in pattern 2 allow more constant solids loading during dry weather flow. This would result in the better estimation of dry weather solids as settled conditions were used to calibrate the dry weather transport model.

Figure 4.112 and Figure 4.114 show the modelled results for the Claverhouse site for fill patterns 1 and 2 respectively. As can be seen, the variation in shapes of the recorded profiles is replicated by the modelled figures. However, it is evident that fill rates are underestimated in the early stages of each data set. This is believed to be related to difficulties in replicating the exact variation of sediment transport throughout the day. The site was characterised by intermittent flows with very few foul inputs. Consequently, the transport of bed material was only observed during the flow pulses. The sampling programme aggregated these flows into segments of 4 hours, making the modelled values only averages over this period. No suspended element was included in this model as a result of the low number of foul connections.

It is also clear from the figures below that, following the initial underestimate, modelled trap rates are overestimated as the modelled line then gradually tends back to observed values. This is understandable, as the lower initial levels of filling will result in a higher level of trap efficiency.

In general it appears that storm trapping volumes are under-estimated. It is believed that this is a consequence of using the aggregated overall trap effectiveness factor. Although the bed load trapping efficiency varies with the incoming trap hydraulics and transported particles, the overall effectiveness factor does not. Consequently, in the latter stages of filling, the overall trap effectiveness factor dominates, as it tends to zero. In reality the storm solids will continue to deposit during receding limbs, regardless of the state of the trap. It is also clear that in the latter stages of trap filling at the Claverhouse site, the re-suspension of trapped material takes place during high flows. This behaviour is not replicated in the model.

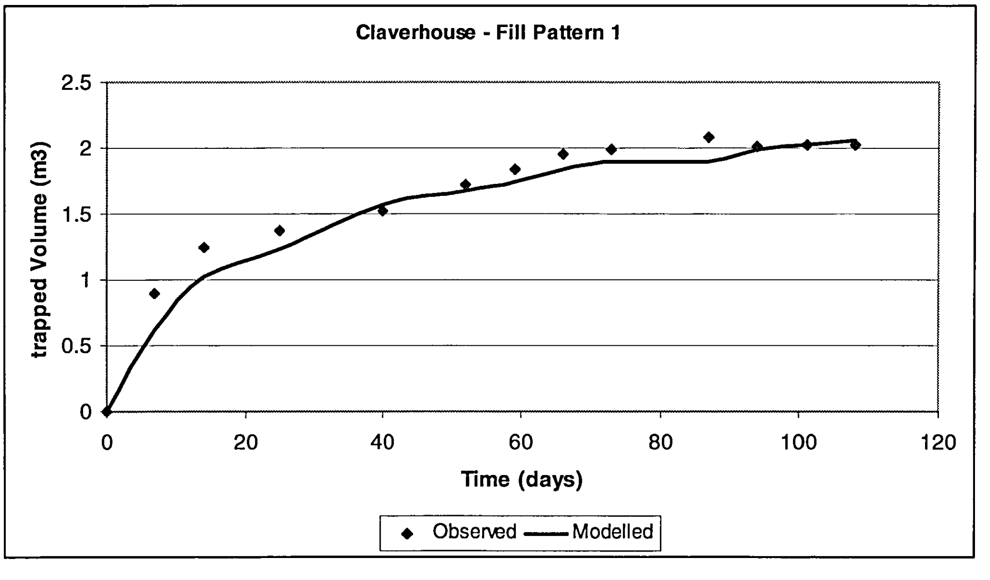


Figure 4.112 - Claverhouse modelled trap filling pattern 1

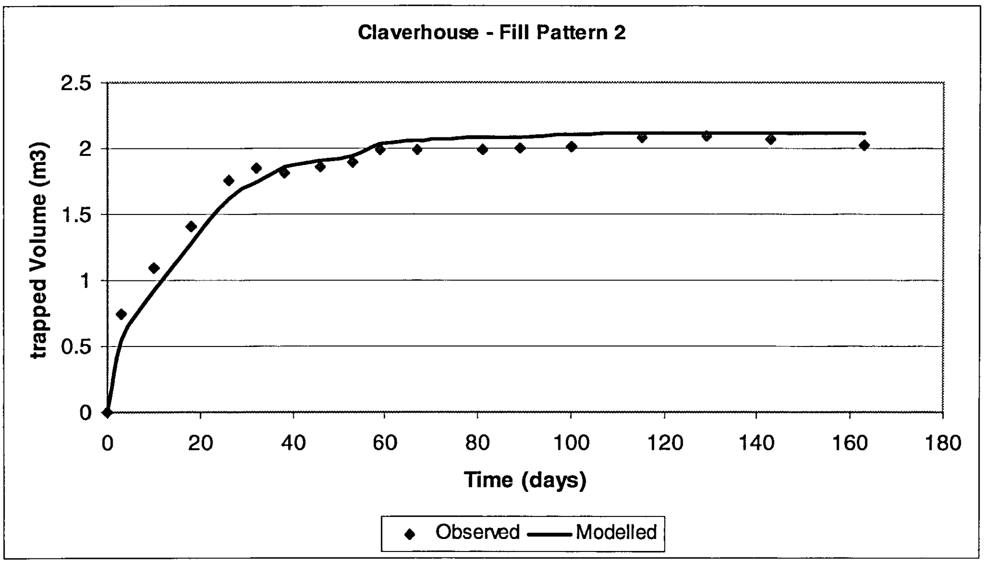


Figure 4.114 - Claverhouse modelled trap filling pattern 2

Figure 4.116 and Figure 4.118 show the results of modelling the trap fill rates for the Constable Street site. In general, the level of fit is acceptable for both data sets. It is apparent that the exclusion of erosion processes only becomes significant in the latter stages of trap filling leading to an overestimate of the final trapped volume for both data sets. This also results in greater deviation from measured values during storms, as although the measured trap volume decreases, the model predicts increased trapping rates as a result of increased solids arrival and changes in sediment characteristics.

Although permanent sediment depth monitors were installed in the trap, the data provided by these monitors could not be used directly for the verification of model behaviour during storm events. This is a consequence of only two monitors being used. The recorded depths were not found to correlate consistently with increases in trapped volume as at different stages in trap filling the recorded readings would vary, with downstream erosion recorded for some events and not others.

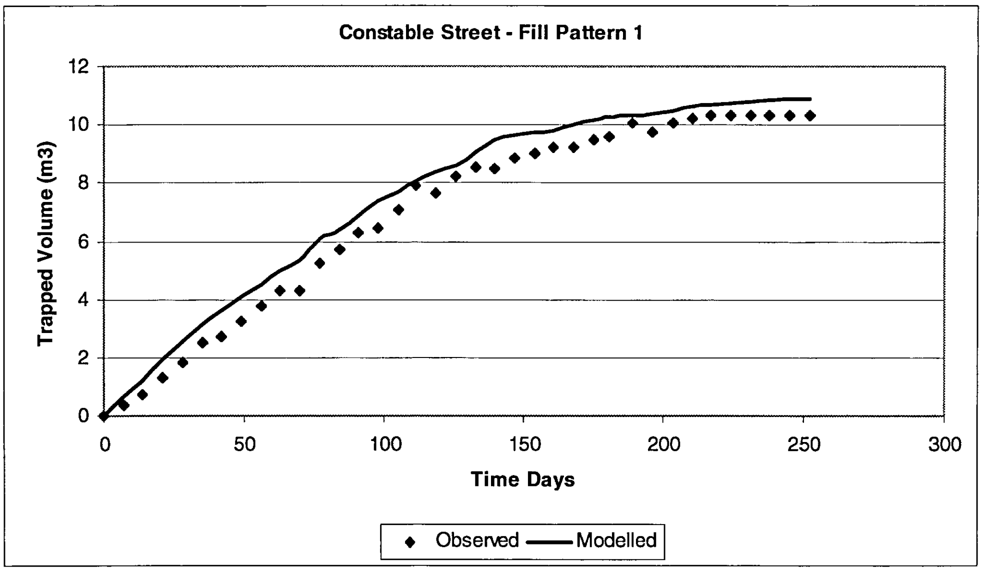


Figure 4.116 - Constable Street modelled trap filling pattern 1

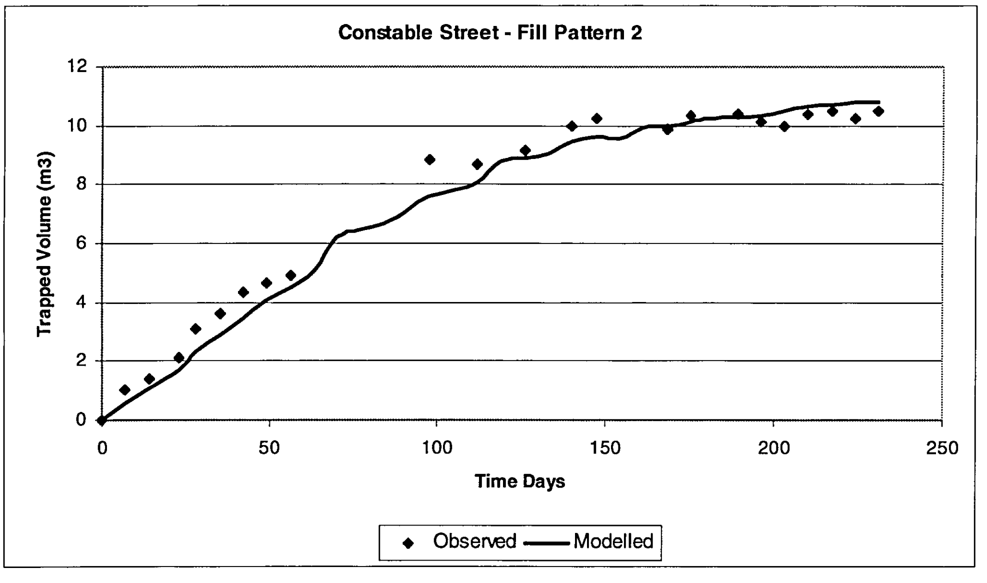


Figure 4.118 - Constable Street modelled trap filling pattern 2

4.10.4 Trap Model Conclusions

In general, the trapping model was observed to perform acceptably at all locations. However, it was hoped that generic design curves for open and partially covered traps could be used to produce the changes in trapping behaviour as the trap geometry changes (due to trap filling). This was not found to be possible as a result of the diversity of conditions experienced at each trap. Each trap model therefore required detailed knowledge of the material type, transport modes and, most significantly, historical knowledge of trap behaviour during filling. It is in this area that a detailed CFD study could provide generic rules for trap performance based on changes in trapped sediment levels and depositional patterns.

A method of designing and planning the maintenance of a sediment trap was devised using the findings of the modelling work and observations of the data collection programme. A summary of this method is provided in Appendix D.

4.11 Combined Hydraulic / Sediment Modelling Tool

One of the principal conclusions of the work carried out during this study has been that, equally important as the individual models, is the way in which all of these models interact. Previous investigations have tended to focus on only one particular aspect of the transport processes. This is correct for the development of a model representing a single process at a single time. However, the processes of sediment deposition are complex and time history dependent. Consequently the development of a deposition model should consider more than just the deposition process.

The field investigations and modelling development detailed previously have highlighted the significance of the loop of influence between sewer system hydraulics and depositional patterns. In addition to this, the hydraulics have a profound effect on the type of material being transported and hence the applicability of any transport relationships used. The purpose of this section of the report is therefore to describe some of the novel aspects of the sediment transport model with respect to the way in which they fit together.

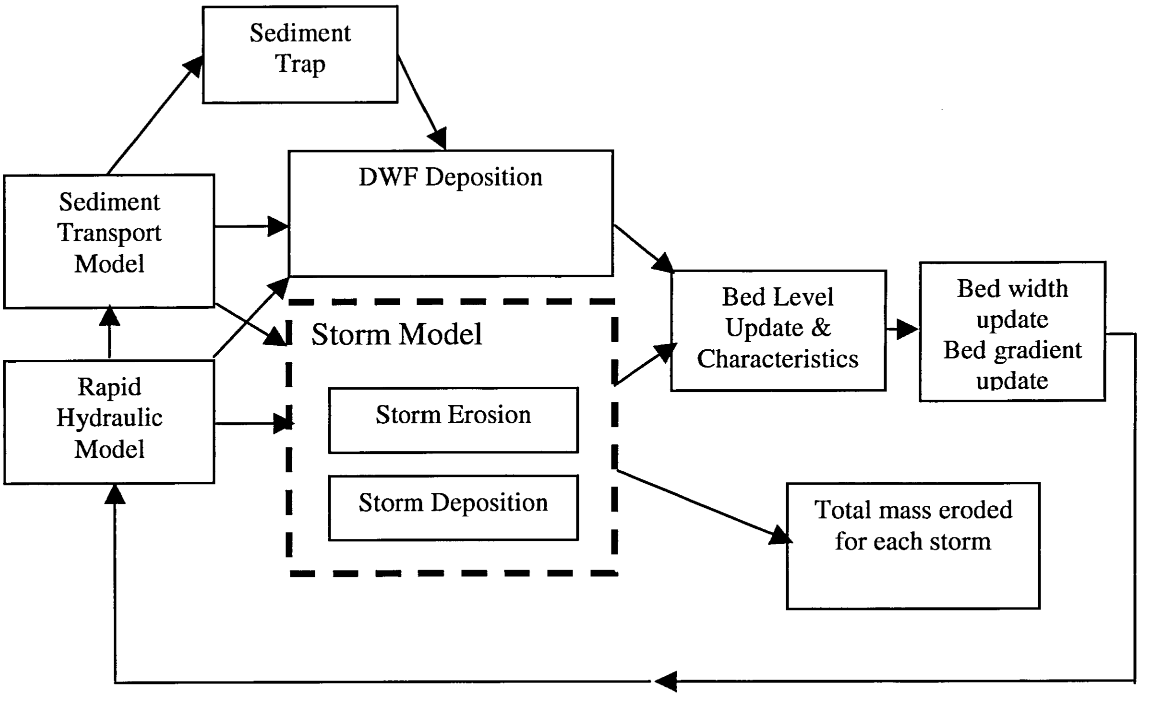


Figure 4.120 - Sediment transport model interaction

Figure 4.120 (above) shows a simplified schematic of how the models link together. The rapid hydraulic simulator is used initially to drive the whole process using inputs of pipe characteristics, dry weather flow, rainfall and sediment bed details (depth, width and gradient). The outputs of this model are essentially flow volume, depth and velocity. These outputs are used in conjunction with the physical details of the conduit to firstly select an appropriate transport relationship using an appropriately sized particle (as described in Section 4.6), then determine the rate of sediment transport present in the conduit. The hydraulic and sediment models are then used to provide input details for the erosion and deposition modules. The input variables for the combined model are shown below:

Variable	Source / Comment
Peak DWF	Measured or model data
Pipe slope and geometry	Measured or model data
Sediment bed bulk density	Sediment density assumed constant 1850 kg/m^3
ζ	Erosion coefficient (0.67)
ξ	Erosion coefficient (0.347)
fine sed %	Engineering estimate
med sed %	Engineering estimate
coarse sed %	Engineering estimate

Table 4.23 - Combined model variables

Currently, during dry weather, it is assumed that no erosion takes place. This results from the fact that the model was developed at a location where dry weather flows were insufficient to exceed bed shear strengths. Consequently, dry weather erosion could not be tested. The erosion model was therefore not included in the dry weather module.

The storm model component contains both erosion and deposition components, with a logical input to determine which model should be used at what time. Essentially, each storm is treated as discrete, and the peak shear stress used to determine a total depth of erosion using bulk material properties. Following this calculation, rates of deposition are determined throughout the receding limb of any event.

The combination of the dry weather and storm modules allows a new bed level to be calculated. This is then used to allow a new array of physical characteristics to be calculated (bed width, roughness and gradient) and used as inputs to the hydraulic model.

All of these calculations are calculated at a timestep of 2 minutes. It is the use of these interactions and decision making logic circuits which enables a greater range of phenomena to be accounted for and contributes significantly to new modelling techniques.

Each of the models was calibrated individually during their initial development. Following their combination together, the combined model was tested against an extensive set of new bed deposit data collected from Dundee's interceptor sewer (as described in Section 3.6.5). Although the same length of sewer was used, the hydraulic conditions within the sewer were notably different from those used during model calibration and development. As a consequence of the intensive sediment studies in the area, a downstream control was identified for the interceptor sewer. This control took the form of a localised sediment mass, deposited at a key junction. It was believed that the removal of the sediments at this single location would have a significant impact on the deposition and erosion processes in the interceptor sewer.

Following the removal of this material, no further cleaning work was carried out in the interceptor sewer. During this period, regular "walk-through" inspections were made at irregular intervals to determine the impact of the new hydraulic regime on sediment levels. The frequency of inspection varied according to weather conditions, with an attempt made to record the impact made by individual events. Detailed bed profiling was carried out using depth measurements at 1-m spacings and sampling at approximately 25-m spacings (samples were not continually taken from the same location). This work was carried out over a total bed length of 145 m. Further details of this data set are contained in Section 3.6.5.

Figure 4.122 shows a summary of the sediment depth data collected during this period. Sediment depths are shown for the date of each walkthrough and can be read using the primary (left) y axis. The profile of the absolute level of the pipe invert can also be seen above the sediment data and can be read using the secondary (right) axis.

As can be seen, the previous equilibrium level of the sediment deposits is reduced significantly following removal of the downstream control. This is especially true from a chainage of 60m onwards. The reason for this can be seen by viewing the long section of the sewer invert over this length. Immediately upstream of this point, a depression in the invert exists which exercises a local control on sediment deposition. As localised sediment deposition was never to be addressed in the modelling work carried out within this study (as a result of the data collection requirements and difficulties in wide scale application), the effects up to this point were ignored. Consequently only the freely discharging conditions from chainage 60 onwards were considered.

The hydraulic inputs were re-calibrated for the depth and velocity calculations to allow for the improved hydraulic conditions and the first recorded walk-through data used as the initial conditions. In this case, as the data were available, the dry weather solids sediment transport model was verified against a new data set. Following these minor alterations to the existing model, the model was re-run to represent the months following the hydraulic improvements using measured rainfall data.

Figure 4.124 shows the uppermost layer of the Fraser sediment model. Within this model hydraulic inputs are modified according to the presence of sediments, and the resulting hydraulic conditions used to select the appropriate sediment transport calculation method. These outputs are then combined and used to activate either the dry weather flow model (deposition only) or the storm model (erosion then deposition). Following these calculations, the bed level is updated along with the pipe geometry, which is then fed back into the hydraulic characterisation process.

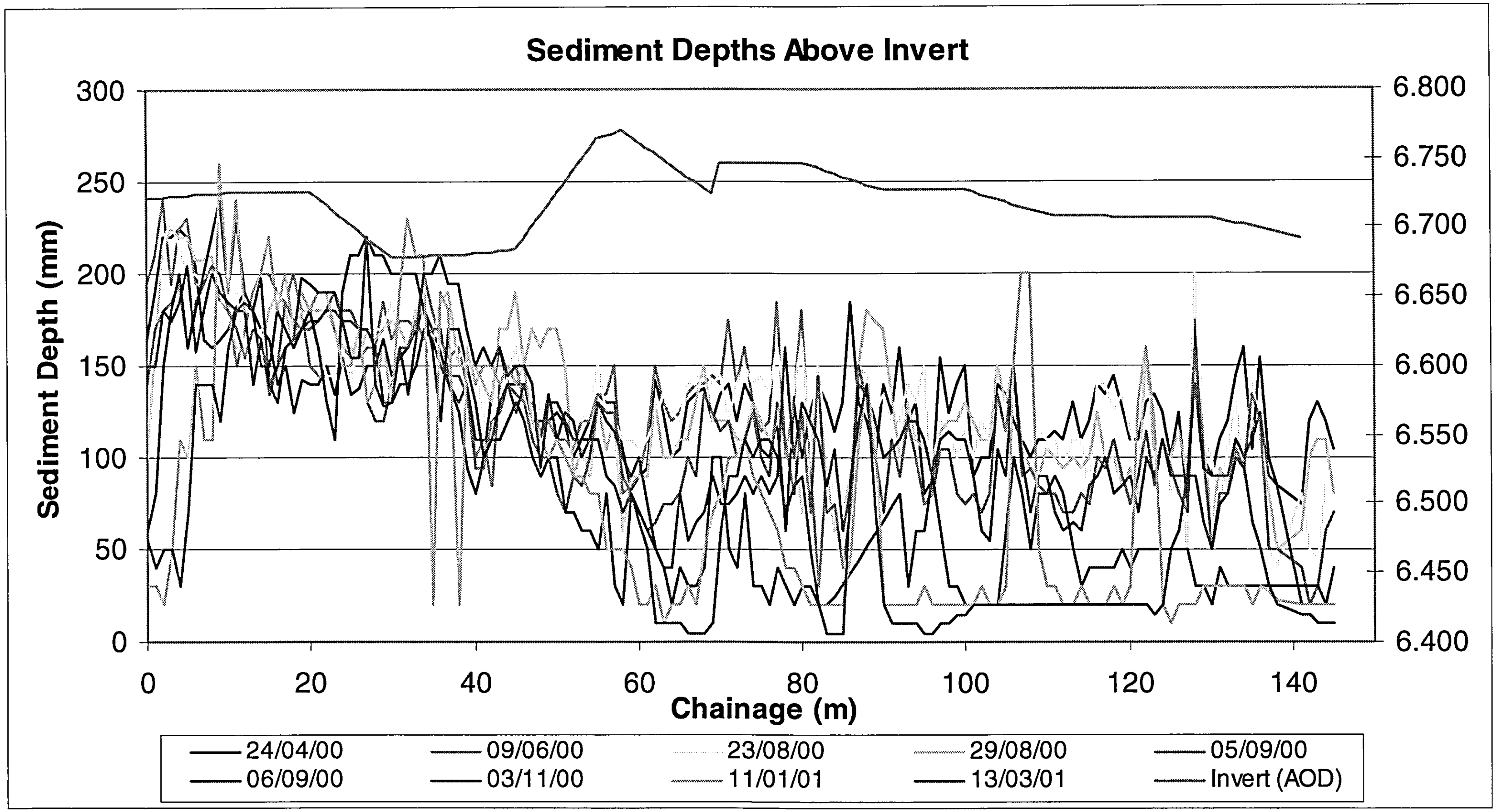


Figure 4.122 – Murraygate sediment bed profiling

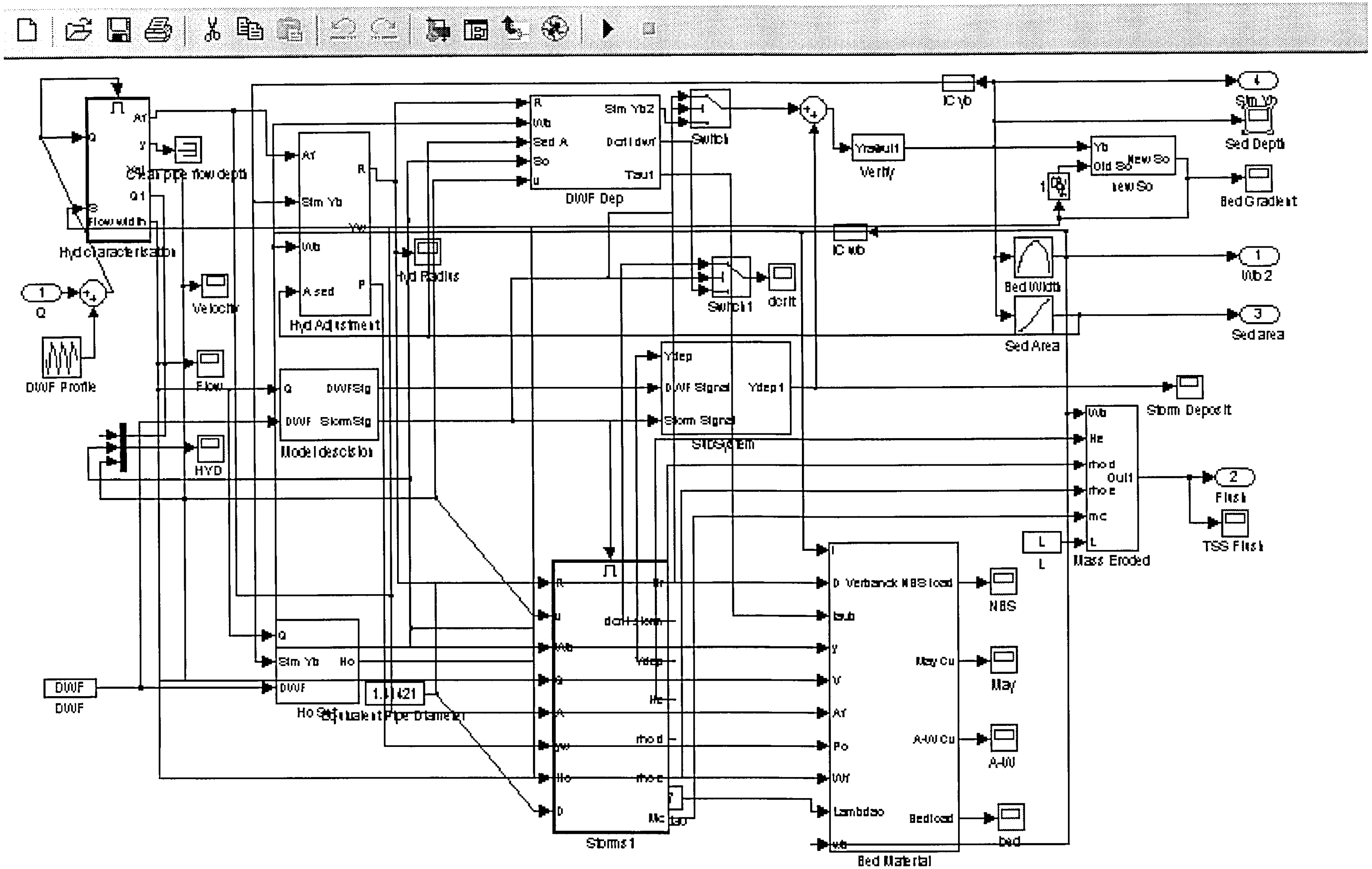


Figure 4.124 - Fraser model (uppermost layer) for sediment prediction

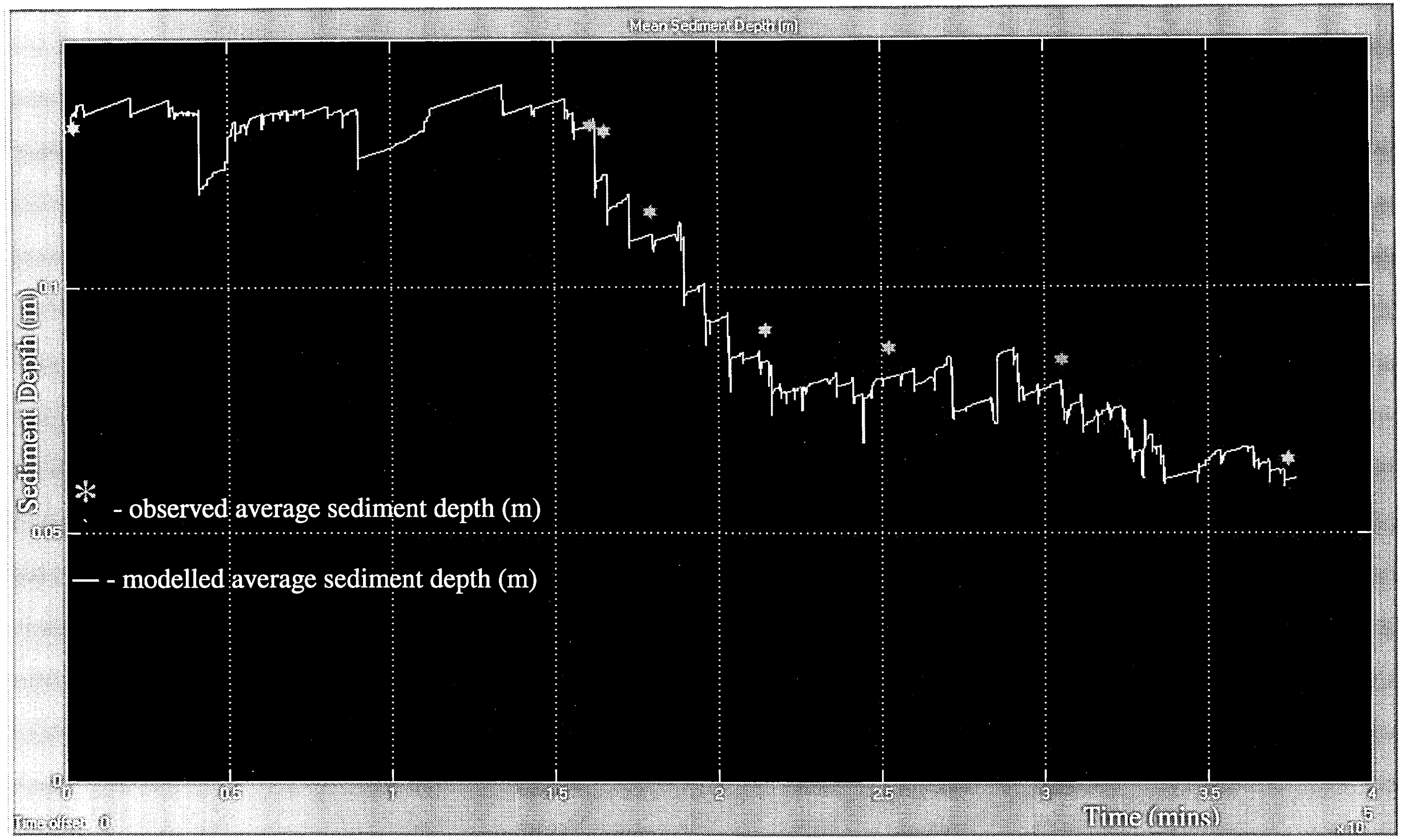


Figure 4.125 - Long-term modelling of sediment levels in Murraygate Interceptor Sewer

Figure 4.125 shows the results of the long-term modelling of sediment levels in the Murraygate sewer over this period of measurement, with average sediment depth (in metres) plotted against time (minutes). The total duration of this simulation is just under 9 months, with a total simulation run-time of approximately 5 minutes. As can be seen, the initial sediment level is a significant 130-mm. This was used as an initial condition in the model in conjunction with the measured bed gradient at this time. In general the patterns of the sediment erosion and deposition are well represented over this period with a minor over-prediction of sediment erosion evident. The final modelled sediment level is seen to be within 4% of the observed value, with general error bands during the simulation of between -5 and +10%. It should be noted that no calibration was undertaken for the long-term simulation, with the various calibration factors taken from the individual event calibration used earlier.

An interesting concept to come out of the analysis of the modelled results is the re-assessment of the notion of the critical shear stress. Traditionally, the attainment of a shear stress greater than or equal to the shear strength of the bed has been seen as sufficient to prevent prolonged sediment deposition. However, this model (based on field observations) is based on the assumption that during the tail of each rainfall event, sediment transport capacities will reduce and transported material will be laid down. Consequently, following every erosion event, a deposition event is seen to occur. This behaviour is evident in the modelled results shown in Figure 4.125, with a sudden increase in sediment levels following each sediment erosion. The significance of this is that in operational terms, the attainment of a critical shear stress based on bed strength may not be enough to cause a net erosion, as the very rainfall event causing that erosion may itself deposit more material than was eroded. This would therefore result in a net deposition. On the basis of this, the critical shear stress becomes that which will impart a sufficient degree of erosion that cannot be readily replenished within the same event. As only the modelled data within this study were of sufficient resolution to test this theory, real conclusions could not be reached regarding the difference between the traditional and net erosion critical shear values. It is believed however that the availability of upstream sediment material will have an impact on the net erosion critical shear stress perhaps making the value site specific.

4.12 Outcomes of Sediment Modelling Work

It is important that any research carried out should be fed back into industrial practices. During this investigation, close contact was maintained with the developers of the two principal sewer flow quality model developers in Europe (Wallingford Software and DHI). These contacts required updates of significant findings and in the case of Wallingford software, the "beta testing" of their sediment models. A report detailing the tests carried out on the HydroWorks two phase sediment transport model can be found in Appendix B.

4.12.1 Improved Commercial Models

As a consequence of the work carried out within this study, a number of recommendations were made to Wallingford software regarding the development of their sediment transport models. The report detailing this work can be found in Appendix B. These recommendations may be summarised as:

1. The build up of the sediment bed is a dynamic process with important interactions between the bed and the hydraulics. It is therefore a severe limitation that the deposits predicted during the simulation do not affect variables such as hydraulic roughness, section shape and hydraulic gradient.
2. The erosion of the sediment bed is modelled at present as a purely granular process. Although sewer sediments are highly variable in nature, some form of erosional resistance can be applied with increasing sediment depth using a shear stress criteria.
3. The length of time that a simulation can be run for is limited by the software via the maximum number of time-steps allowed. If this feature is desirable for standard simulations, perhaps a long term simulation mode could be introduced to aid sediment deposition prediction. This mode should allow a variable timestep for dry weather and storm conditions.

4. The reviewing of deposition results is difficult using a graph for each pipe. A colour coded plan view with a key for different levels of deposition (similar to flood volumes results) would make analysis very simple for the user.
5. It is believed that at present, the greatest potential use for the sediment transport module is in sediment deposition prediction. It also clear that this can be combined (using long term simulation) to predict flushes of material, the current approaches however are not suitable for this long term application.

Since these recommendations were made, a number of other researchers have added to the work carried out in assessing commercial sewer sediment transport models (Boutteligier, Vaes, Berlamont, Margetts, and Long). The combination of these investigations has resulted in a number of alterations being made to the initial HydroWorks 4.0 two-phase model. To date, these changes have culminated in the issue of InfoWorks v5.0. The use of the InfoWorks platform allows modellers to utilise a variety of functions linked to Geographic Information Systems (GIS). These functions allow some of the result interpretation issues highlighted in the initial HydroWorks assessment to be addressed. The changes made to the modelling software may be summarised as:

- A feedback option is provided to allow changes in sedimentation patterns to affect conduit hydraulics (and hence in turn influence future deposition);
- The implementation of a continuous simulation mode which enables the time step to be varied according to dry weather or storm conditions;
- Additional sediment transport relationships have been made available - Velikanov (Zug. et al., 1998) and KUL (Bouteligier et al., 2002);
- Limit of level of sediment deposition according to flow depth removed;
- Sediment carrying capacity limited to 2 g/l/sediment fraction;
- Dimensionless grain parameter limited at lowest level to 1.0;
- Maximum surface sediment erosion rate definable;
- Option provided to match run-off and wash-off model processes

The work carried out as part of this study has had a clear and definable input to industrial practices in the first three of the model alterations detailed above. It is hoped that the further testing work carried out within this thesis will contribute further to model improvements.

The work carried out as part of this study has had a direct link to industry and has influenced the development of commercially available models. The most significant of these impacts has been the feedback of depositional patterns into the system hydraulics and the development of continuous simulation options.

However, another of the principal objections to the InfoWorks sediment transport model continues to exist. This involves the use of a granular analysis to predict erosion of cohesive material. Further work carried out here and at the Katholieke Universiteit Leuven has identified further problems with the current software release (Bouteligier et al., 2002). It is anticipated that this work will continue to refine the model towards a reliable state.

The current InfoWorks model has been tested for the reproduction of depositional patterns in a test catchment and has been found to perform significantly better than previous release versions. It was also found that model performance could be enhanced through the use of improved model initialisation.

Chapter 5 : Application of Modelling Methods

5.1 Introduction

The techniques developed throughout Chapter 4 were combined and used in conjunction with the data collection experience described in Chapter 3 to produce a range of methods suitable for the management of sewer sediments.

These methods were applied to an entirely new catchment to determine the applicability of the approach and devise a solution to a sediment related problem. The catchment area selected was that of Lugar, in Ayrshire, Scotland.

The catchment of Lugar is part of the Cumnock drainage area and is located approximately 15 km to the east of Ayr. The catchment covers an area of approximately 24 ha and contains a total population of 221. The catchment has a significant watercourse, the Lugar Water, which flows from north-east to south-west, which in this area has been classed as salmonid and of medium amenity. The upper areas of the catchment are initially steep, draining to a relatively flat trunk sewer. This trunk sewer discharges to a pumping station via a complex arrangement of overflows. This arrangement had been deemed unsatisfactory.

Investigations in the catchment have highlighted the significance of sediment deposits. Deposits are seen to occur in localised areas at the head of the catchment and more significantly, along the length of the trunk sewer. As a result of the large influence that these sediments have on the operation of this small catchment and the availability of good CCTV coverage in the area, this catchment was determined as suitable for assessment of the method.

5.2 Method Applied

The methods developed in Chapter 4 were applied under a slightly modified framework. This was a consequence of the increased durations of continuous simulations offered by the latest versions of InfoWorks; the small size of the catchment (making run times shorter); and the desire to implement some of InfoWorks GIS capabilities within the modelling strategy.

Under the modified strategy, the rapid hydraulic simulator was not used to produce flow volumes. The continuous simulation of a stochastic annual series was used to provide hydraulic inputs directly to the hydraulic characterisation phase. It should be noted that this offers greater flexibility in the range of solutions that can be considered as small operational alterations to flow regime such as pumping levels can be tested using the detailed hydraulic model. Under the hydraulic characterisation phase, the depths and velocities of the flow were determined. These elements were not determined using InfoWorks as a result of the important links between the deposition process and hydraulic characterisation as described in Chapter 4.

In addition to this, the InfoWorks sediment transport model (KUL) was used to determine sediment transport rates in each of the pipes. These data were used as inputs to the sediment location model transport model.

These minor modifications demonstrate the flexibility of the approach developed according to the circumstances of the test case and software developments. The modified structure of the method applied is shown below.

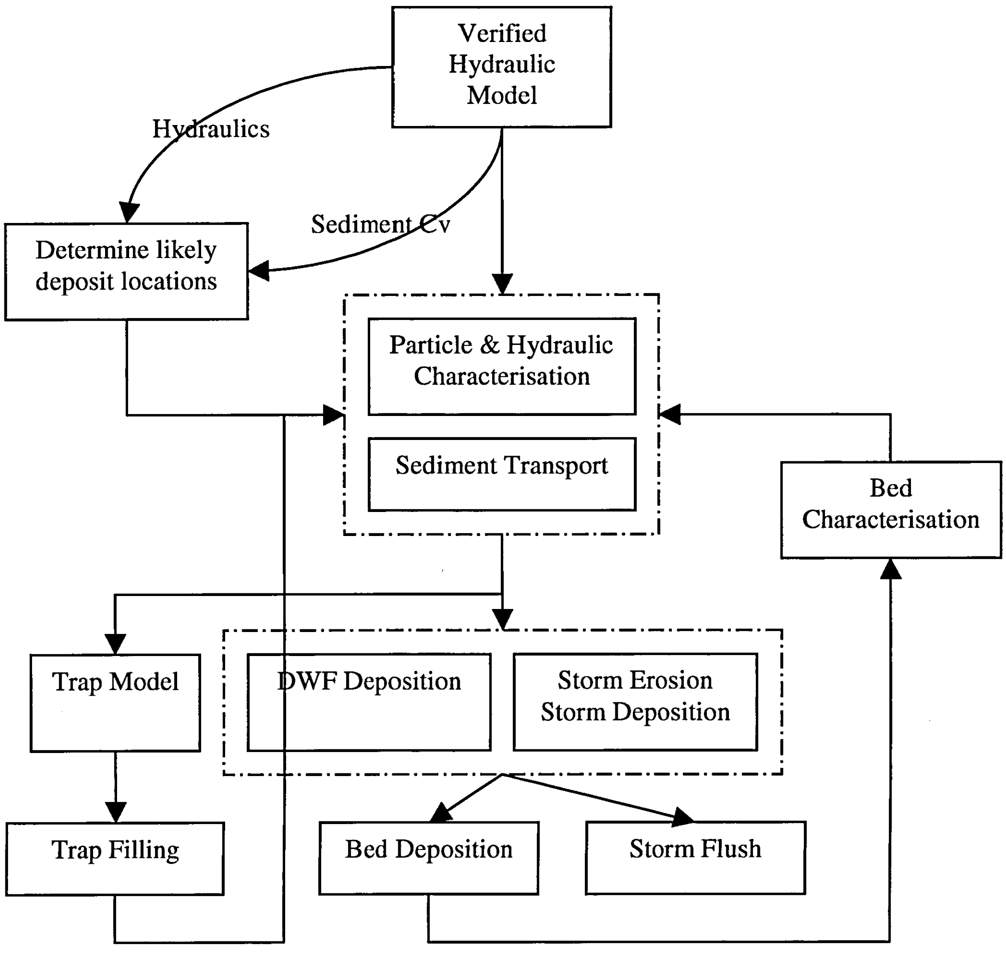


Figure 5.1 - Modified sediment model structure

5.3 Sediment Location Results

Figure 5.2 shows the observed pattern of deposition in the catchment recorded during an extensive CCTV survey. Although not all pipes were surveyed, a good coverage of pipes was achieved by the survey. As the pipes in this part in the catchment are generally small (150 mm to 400 mm) a relatively low level of sedimentation was deemed significant in a large portion of the catchment. Consequently, for the observed data, deposits of more than 15 mm of sediment were deemed significant and are shown below. However, the most extensive area of deposition was found in the principal trunk sewer.

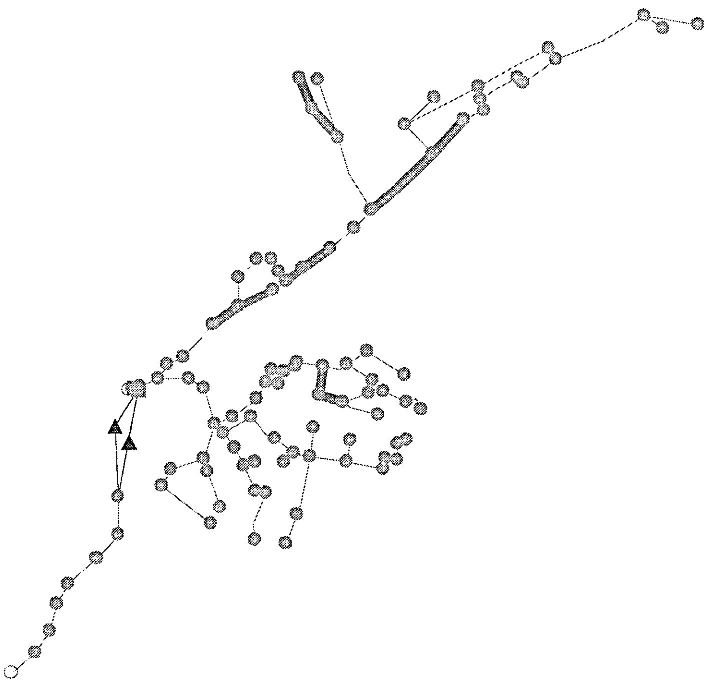


Figure 5.2 - Observed deposition location in the Lugar catchment

The sediment location model was applied using the Mosqito default sediment characteristics as described in Section 4.4 and using critical storm shear values of 2.5 N/m^2 and 9 N/m^2 for storm 90 and 15 respectively from the ranked annual time series.

The GIS functionality of InfoWorks was found to ease the labour intensive nature of the process. Automatic mapping of peak shear stresses and the tracing of upstream and downstream links was found to be particularly useful.

Seven pipes were identified to be at risk from sedimentation. Five of these pipes were located in the trunk sewer identified as exhibiting sediment problems by the CCTV survey. The remaining two pipes were located further up the network. A review of these locations reveals the hydraulics to be affected by high headlosses at junctions involving several incoming pipes at angles approaching 90^0. One of these pipes was identified as being affected by sediments during the CCTV survey.

Figure 5.3 shows the results of this analysis. This figure should be compared to Figure 5.2. As can be seen there is a good correlation between observed and predicted data, with the principal areas of the trunk sewer identified. The exact extents of the deposition were not replicated as the procedure does not take into account the effects of the deposits on hydraulics and therefore the spread of the deposition into adjacent areas. Additionally one of the most upstream areas with the most severe deposition was not picked up as the condition of upstream dry weather erosion was not met and as the pipe is at the head of the system no storm erosion upstream was identified.

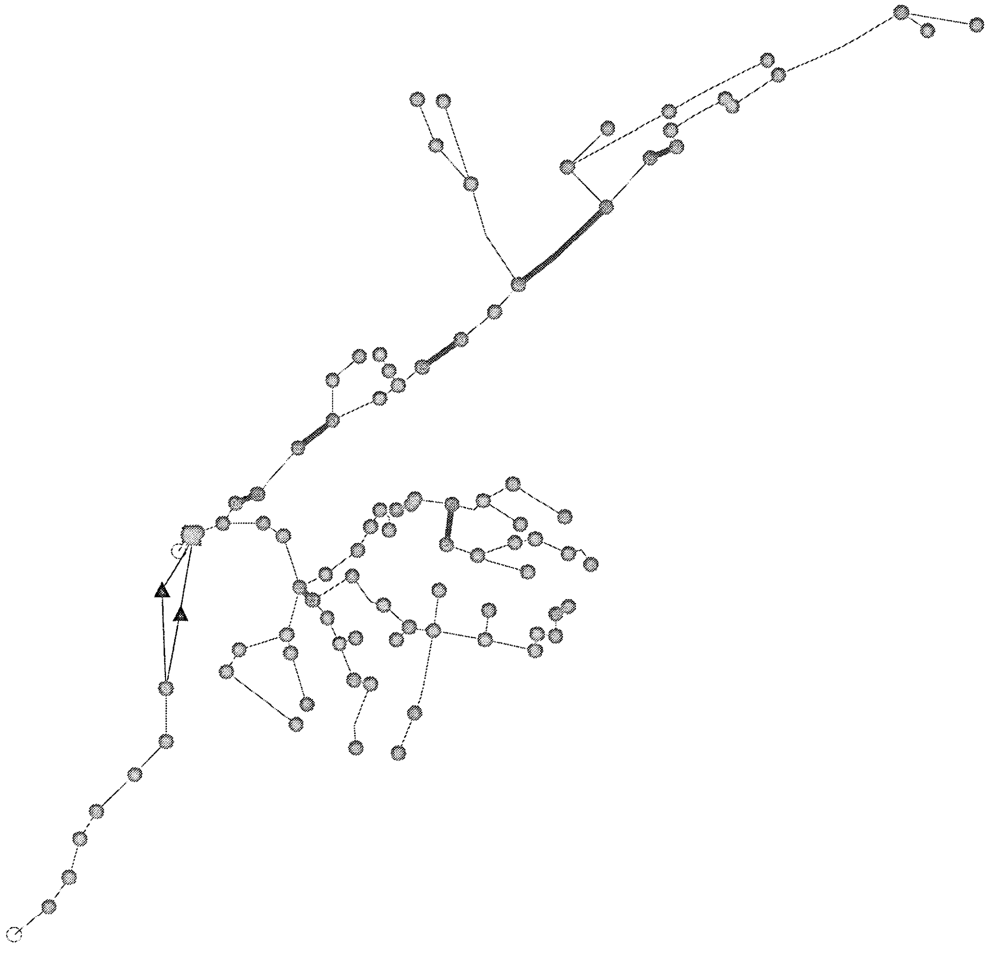


Figure 5.3 - Sediment location model results

5.4 Sediment Quantity Results

In order to assess the scale of the sediment problem and devise a suitable sediment management strategy, predicted sediment quantites were determined for the key location.

As the trunk sewer was identified as the principal location of sedimentation, the sediment quantity model was used to estimate the rates and depths of deposition experienced at this location.

The stochastic annual series was run through the InfoWorks model using continuous simulation. The resulting flow volume data were then extracted for the inlet to the longest length of trunk sewer most affected by sedimentation.

Flow data were extracted for the upstream node for this section and were used at 2-minute timesteps to drive the sediment transport and deposition models. The assumed sediment bed bulk density was kept constant at the same value used during model development (1850 kg/m^3).

A sediment depth of approximately 50 mm was predicted following the initial run with a simulated duration of 1 year. However, it was unclear from this run whether a steady state sediment level had been reached. Consequently, the hydraulic input was repeated to provide 2 years of continuous simulation. The output of sediment depth for this simulation is shown in Figure 5.4, with sediment depth plotted on the Y-axis against time in minutes. A final sediment depth of 62 mm is predicted. This compares favourably with sediment depths of between 50 and 60 mm recorded within the trunk sewer during the CCTV survey. As 60 mm represents a significant restriction within a 300 mm pipe and a considerable pollutant store, a mitigation measure was devised.

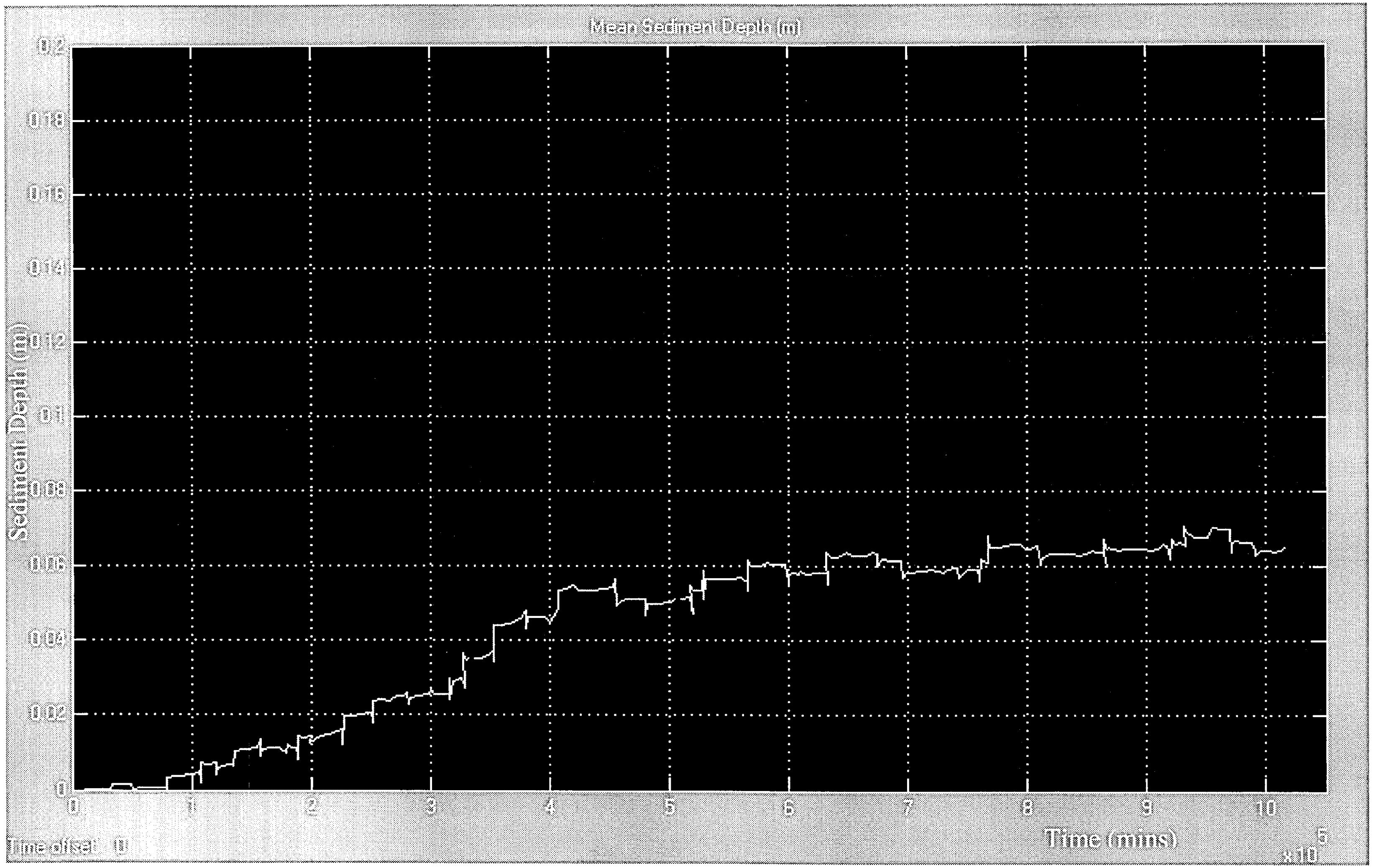


Figure 5.4 - Lugar trunk sewer sediment level prediction

5.5 Sedimentation Mitigation

In the case of the Lugar catchment, the role of the deposited sediments in the trunk sewer as a contributor to overflow pollution was of the greatest concern. Consequently it was important that there should be no sediments available for re-suspension. The types of solution appropriate were therefore:

- Hydraulic improvements at the pumping station;
- Interception and storage using a sediment trap;
- Automatic flushing.

The effect of improving pumping station operation was found only to improve system hydraulics in the lower reaches of the system. It was also perceived that the potential use of flushing gates would be ineffective as they would not flush to a free outfall or area of higher sediment transport capacity. Consequently, the most appropriate method in this case was determined to be that of trapping sediments. This allowed the potential of using the trap model developed in Section 4.10 to modify the sediment inputs to the depositing pipe length and gauge the effects of the trap on bed development.

5.6 Mitigation Testing

The model was re-run using the sediment trap model described in Section 4.10 to modify the sediment inputs to the pipe deposition model. The effect of the trap was to reduce the incoming concentrations of sediment into the trunk sewer and hence reduce the trunk sewer's sedimentation rate. A trap volume of 10 m^3 was used in conjunction with the efficiency curve derived from the Constable Street sediment trap (as it was of similar volume).

Figure 5.5 shows the predicted build up of sediment in the trunk sewer following the installation of a sediment trap upstream of the deposit locations.

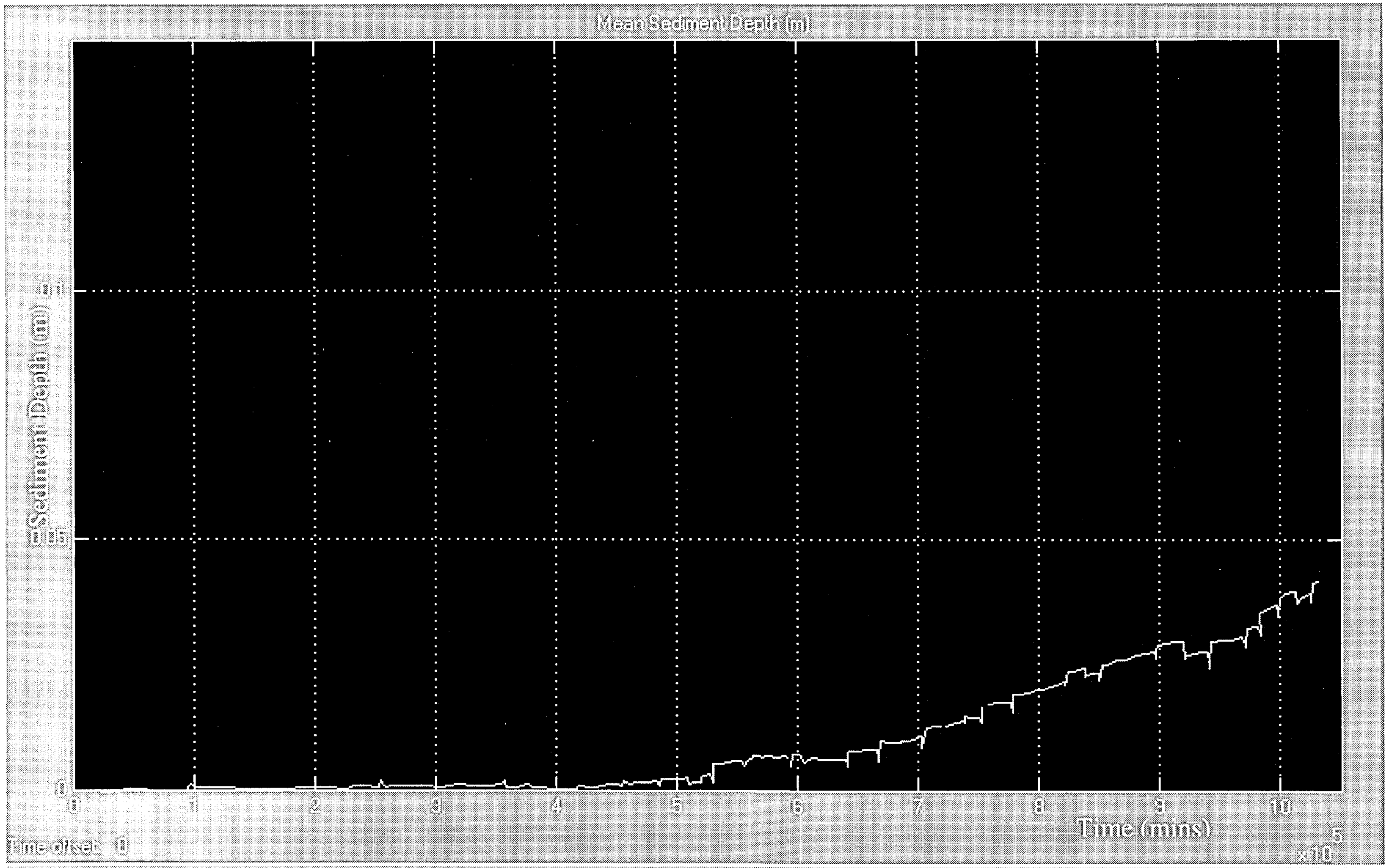


Figure 5.5 - Predicted Lugar trunk sewer sediment levels following trap installation

As can be seen in Figure 5.5, sediment levels are kept to a minimum when the trap is used to reduce sediment loadings. As the trap fills, over the first year of simulation, there is an insufficient volume of sediment arriving at the depositing pipe to create any deposits of significance. Once the trap starts to lose its trapping efficiency, greater sediment arrives and is consequently deposited in the pipe. However, the accelerating rates observed prior to the use of the sediment trap are delayed and reduced as a result of the diminishing effect of the sediment trap. By the end of the simulation, the trap is completely full and subsequntly the rate of pipe deposition increases at a similar rate to that predicted previously. At the end of the 2 year simulation duration a sediment depth of 46 mm is predicted. Although this is significantly less than that previously modelled, it should be noted that an equilibrium level is not yet reached.

As the principal concern at this location was the role of sediments as a pollution source, ideally no sediments should be present on the trunk sewer. The results of the model indicate that the use of a 10 m^3 sediment trap cleaned at a five month frequency would maintain sediments at a negligible level in the downstream trunk sewer. However, there would be some sedimentation as a result of local connections along the length of the trunk sewer that are not intercepted by the trap.

5.7 Critical Evaluation of Lugar Application

The Fraser sediment transport model and the techniques developed within this thesis were applied to the Lugar test catchment. Although this catchment provided a high level of available data for a typical catchment, time varying sediment bed data and sediment transport data were not available for verification purposes.

However, it is concluded that the methods proposed in this thesis are able to predict accurately both the locations and ultimate depths of sediment in the Lugar catchment. Further use of the modelling techniques allowed an effective method of sediment control to be devised and the prediction of a minimised maintenance scheme for the 10 m^3 trap with a five month cleaning frequency. In this way it was demonstrated that the techniques can also be used to successfully manage a sediment problem in order to prevent pollution incidents and ensure the maximised hydraulic capacity for the Lugar catchment.

Chapter 6 : Conclusions & Recommendations For Further Work

At the outset of this programme of research, a review of current literature revealed that the current approaches to sediment transport (deposition, erosion and movement) were inadequate for wide-scale and long-term application. The identification of these inadequacies and the development of measures with within the Fraser method provides engineers with improved tools and methods for sediment prediction.

The methods set out in this study address for the first time the complex interactions between sediment and hydraulic processes over prolonged durations. In addition to this, the previously highlighted limitations of the use of a single particle size over an entire catchment and inappropriate model application have been addressed. It is anticipated that these important innovations should be assimilated into commercially available sewer flow quality models as a result of concerns over their current approaches and their increasingly wide-scale use.

6.1 Data Collection

A wide ranging programme of data collection was undertaken to assess the characteristics and behaviour of transported, deposited and trapped sediments. As a result of the difficulties and high costs associated with the running of a long-term sewer data collection programme, there is a general dearth of data collected over prolonged durations. This is particularly true in the UK, with the majority of long-term studies emanating from France. The data from this study of sediment trap fill volumes, sediment bed development and sediment bed erosion are therefore rare and should be considered as valuable for extending sediment knowledge to long term behaviour.

Although, in the main, standard methods of data collection and instrumentation were used (e.g. flow logging and suction sampling), other methods were developed and tested for application to a sewer data collection programme. Most significant amongst these were the fixed sonar units and the cryogenic corer. These instruments were developed "in-house" and tested within the laboratory to provide details of trap filling that were previously unavailable.

Sediment transport measurement confirmed the existence of a transport phase similar to that termed "near-bed-solids" at the site monitored in Forfar. This had significant implications for the deposition and trapping of sediments at this location. A concentration profile increasing with proximity to the pipe invert was noted at all sites.

Long-term sediment bed data were collected over a period of approximately nine months. This tracked the gradual reduction of sediment levels in the Murraygate sewer. No other such data set is known to exist.

Three traps of varying characteristics were used in this study to provide data over as wide a range of hydraulic and sediment conditions as possible. The recording of sediment transport rates at these sites, using bed traps and suspended solids profiling, highlighted ranges of sediment transport phases and the difficulties in representing sediment transport phenomena with a single set of equations (as is currently the case).

The use of the unsuitably located Forfar trap afforded the opportunity to modify its configuration in an attempt to enhance its performance. The trap is located in an area subjected to velocities in the order of < 0.1 m/s. Consequently, a high deposition rate occurs both in and around the trap. The approach of partial covers was used to attempt to limit the type of material retained within the trap as this approach has been successfully applied elsewhere. The effect of this work was to decrease the effectiveness of the trap as a sediment management measure. It is believed that, although marginally less material was entering the trap, as a result of the covers, material that would otherwise have been washed out from the trap could not be. Consequently, the trap was found to fill faster with low density, "near bed solids" type material. This work highlighted the importance of good site selection in the design process, as even with the use of CFD modelling and an extensive knowledge of the local sediment characteristics, the fundamental problem of poor site hydraulics could not be overcome. The findings of this ionvestigation propose that invert traps are only located in sewers where the bedload transport phase dominates. It is recommended that this is ascertained through the use of flow surveys for potential sites and the calculation of the sedimentation parameter (η).

6.2 Modelling Tools

The principal focus of the models developed during this investigation was not the development of a new set of equations for transport prediction, but rather to overcome the shortcomings and limitations of the wide range of currently available numerical methods (see Section 2.8.6). Central to this objective was the observation of how sediments behave over long time-scales and at wide ranging locations. These observations indicated that it is the interactions of the various processes and not the modelling of the actual processes themselves that have been neglected in previous studies. Consequently, the Fraser sediment model was developed within the SIMULINK programming environment and was tested against a variety of data sets. The model comprised various submodels linked together through a feedback of information that allowed each of the sub-models to alter over time.

The initial assessment of potential sediment locations for further investigation was carried out using the analysis of flows generated by a verified HydroWorks model under both dry weather and storm conditions. Following the initial investigations of Gent and Orman (Gent & Orman, 1991), their method was tested and modified for wider applicability. The application of this method using HydroWorks was found to be relatively laborious. This may hinder the future widespread use of such methods. However, the increasing use of programmable GIS applications will reduce the manual mapping required when using only HydroWorks and a spreadsheet model.

This overlap of GIS and analytical techniques would provide additional functionality and should be further investigated. The model itself was found to produce generally accurate results when compared to an extensive set of data for the City of Dundee. Known localised deposits were on occasion missed by the analysis. This was principally because the nature of the causative elements was not hydraulically modelled (e.g. unknown discontinuities), and the fact that only a single characteristic particle is used in the model.

A unit hydrograph method was tested and used for the conversion of rainfall inputs to flows. The method generally performed adequately. However, the model was found to over-predict volumes and peak flows at the catchment outlet for a weekly event. Efforts made to rectify this through the calibration of subcatchment runoff were found to result in the under-prediction of flows for larger events and also for other parts of the system for the weekly event. As the principal focus of the model was to produce flows for key deposition locations, and these were unaffected (maximum 15% error), the over-prediction of flows at the catchment outlet for the smallest event was accepted as a limitation of the model. Investigations into the reasons for this overestimation revealed a number of potential contributing factors:

- Increased influence of rainfall losses in the smallest events being underestimated;
- Unsuitability of the peaked unit hydrograph profile to produce the flat weekly event profile under convolution;
- Misrepresentation of bifurcations at lower flows.

The misrepresentation of flows for small rainfall events is an area that must be resolved before the unit hydrograph approach can be widely used for rapid operational assessment. This is therefore an area of recommended further work.

The conversion of these flows to velocity and depth inputs for sediment transport models was found to produce good results. Very little deviation from HydroWorks or measured data was observed.

A review of existing models for determining quantities of sediment deposition led to the development of the USEPA method (Pisano et al., 1979). The initial format of the equation was retained with the inclusion of a bed width factor to allow for changing influences as any sediment bed forms. The model was developed using a historic data set (Coghlan, 1997) for one sediment deposit location. The overall modelled deposition quantities and patterns were found broadly to match observed sediment bed depths. However, the principal limitation of the original Pisano model was that no erosion effects were represented. This limitation was addressed during the development of the Fraser model.

Within the Fraser model, sediment erosion was represented using the Wotherspoon cohesive erosion model (Wotherspoon, 1994). A dearth of suitable cohesive erosion models was revealed during the literature review, although the Skipworth model (Skipworth, 1996) was also considered. The Wotherspoon model was modified slightly to remove the calculation of negative erosion (deposition) at shear stresses less than the critical erosion value. Following this minor change, the model was found to predict closely maximum erosion depths to a level of accuracy comparable with that previously reported by Wotherspoon (Wotherspoon, 1994). It is recommended that the suitability of erosion models to widespread application should be regularly assessed as a number of cohesive and mixed particle size studies are currently underway.

The literature review of sediment transport methods highlighted a number of previously identified limitations. In the past, attempts have been made to be overcome these limitations through the development or modification of methods for universal applicability to sewer sediments. The diversity of sewer sediments and their processes has continued to provide a stumbling block for a single method. Consequently, within the Fraser method, a range of transport models are available and are called into operation according ambient hydraulic conditions at any given time. The approach draws on the experience of CIRIA Report 141 (Ackers et al., 1996) and extends it to three transport modes determined by hydraulic and sediment inputs. The models used to represent the transport of near bed solids, dry weather solids and storm solids were Verbanck (2001), May (1993) and Ackers and White (1991) respectively. The previously extensively tested models of May and Ackers and White were not re-tested. However, the Verbanck approach was successfully tested against low-density suspension data taken under low shear stress conditions. Further details of this work are contained in Section 4.6. It is recommended that the assumed change points of model suitability (based on development data) should be reassessed through detailed laboratory studies. These studies should use "real" sewer sediments and should attempt to observe behaviour over a wider range of flows and material mixes than previous studies. This would enable transport phase change points to be defined more accurately and hence ensure the selection of the most appropriate technique.

In addition to developing the intelligent selection of sediment transport formulae, routines are included within the Fraser models to allow the characteristic size of deposited particles to be estimated. An initial comparison to deposited characteristics at three sites revealed a good correlation. Consequently, this approach was extended in an attempt to modify the assumed characteristics of material in transport as a result of deposition and erosion processes. These routines are largely untested. It is proposed that the inclusion of routines such as these in commercially available software could have a substantial improvement on "calibration" methods. At present, a quality model user has significant control over a wide variety of characteristics. This has led to a wide variety of calibration methods often using unrealistic characteristic parameters. The inclusion of an automatic particle characterisation routine removes the scope within these models for curve fitting rather than calibration.

The interacting processes of the various sediment transport sub-models were analysed. Most significantly, the loop of influence between sewer hydraulics and depositional patterns was highlighted. In addition to this, the links between system hydraulics, the appropriateness of any given transport model and the influence on particle size transported were explored. This analysis resulted in a combined sediment transport model allowing these interactions to be represented.

The combined model was then tested, with no additional calibration, against an entirely new set of data collected from the Murraygate sewer under a hydraulic regime in which shear stresses were greater than those experienced during previous data collection periods. The combined model performed admirably and represented the reduction of sediment levels over a period of nine months to a high degree of accuracy (-5% to +10%). Further details of this work and results figures are provided in Section 4.11.

The modelling tools allowed various influencing factors to be tested. During deposition it was clear that, at different stages of bed development, different elements dominate the depositional processes. This is shown in measured data sets as an "S" shaped curve of sediment depths plotted against time. The principal influences highlighted were those of the changing bed gradient and roughness effects through the influence of the bed width.

The concept of critical shear stress to prevent deposition under an eroding regime was investigated. An examiniation of the behaviour of the model and field data clearly showed that the critical erosion to prevent sediment deposition should not be that which is able to initiate the erosion of particles, but instead, that which shows a net erosion after storm deposition has taken place. On the basis of this assumption, it is likely that the critical shear stress is likely to vary spatially (even under similar hydraulic conditions) and for varying events. A detailed laboratory study mimicking the changing flows and loads of observed storms is therefore recommended to assess the detailed mechanisms of this behaviour.

Two sediment trap models were developed. An initial model was tested against a data set from the Meadowside silt trap in Dundee. This simplified approach used sediment transport theory and the assumption that the trap retained all material in transport near the bed. The simple model produced results that exceeded initial expectations and showed merit in developing the approach to account for further processes. A more advanced model was subsequently developed using the findings of fieldwork investigations and also those of a parallel laboratory and CFD study in the University of Sheffield (Buxton, 2003). The dominant influencing factors were identified and included in the model, with the model predominantly based on an assessment of the ability of the bed material to jump over the trap. It is clear from the findings of the sediment transport models in this study, that the characteristics of material in transport will alter as flow behaviour changes. Consequently, a method was devised to assess the bed particle most able to bypass the trap. This particle was found to be best characterised by a sedimentation parameter of 10 (see section 4.8.2.2).

In order to represent the influence of the inputs of suspended material, the findings of the CFD study were used directly through the application of efficiency curves. The reduction of the ability of a trap to retain material as a result of changes to its geometry during filling was represented using purely empirical means. These efficiency curves were found to vary significantly at each site resulting in the use of specific relationships. As this efficiency is believed to be the dominating factor of the model, the shapes of the observed and predicted fill rates at each site were well matched.

The suitability of the combined model to manage sediment problems was tested on the small catchment of Lugar. The existing scenario was initially tested and was found to accurately replicate the observed extents and levels of deposition. A strategy for sediment management for the catchment was devised using the trap model, and a maintenance regime was determined to allow the trunk sewer to remain sediment free.

6.3 Summary of Recommendations

The work carried out during these investigations has resulted in a number of recommendations being made. These are summarised below:

- It is recommended that the Fraser method should be used to determine sediment transport rates within drainage systems and predict the locations, extents and depth of sediment deposits. It is further recommended that these tools are used within the strategy set out in Chapter 5 to manage any predicted or observed sediment problems.

- No single transport formula was found to represent sediment transport adequately under the required range of flows and particle types. It is therefore recommended that this traditional approach is replaced with the method used within the Fraser sediment model. Within this method the model automatically selects and applies the most appropriate numerical technique according to the ambient hydraulic and particle conditions.
- It has long been noted that the selection of a single characteristic particle restricts the applicability of sediment transport methods. The techniques investigated in this thesis recommend only limited modeller control over sediment characteristics and that future models hydraulically sort particles added to the model to represent typical surface and foul inputs.
- The observation of model behaviour highlighted the erroneous nature of a critical shear criterion (particularly during storm weather). It was found that each erosion event was followed by a deposition event. In some cases this deposition event was found to exceed even significant erosion depths. It is recommended that the critical erosion should be the shear stress that results in an overall net reduction in sediment levels. This value is likely to vary according to location.
- The feedback of updated data allowing for the effects of calculated sediment deposits on the hydraulic depth/velocity calculations was found to have a significant impact on the calculation of deposition rates. This phenomenon should always be included and should allow for changes in cross-sectional properties, roughness changes and modifications to the average bed gradient.
- The work carried out predicting the potential locations of sediment deposits demonstrated the accuracy and value of the method, and highlighted the increased efficiency of the process when using GIS based tools. It is further recommended that the default particle characteristics of Gent & Orman (1991) should be used to define solids for the dry weather analysis.

- When using the unit hydrograph method, the time of concentration (Tc) for the event of the highest intensity to be applied should be used to derive the Tc unit hydrograph method. This approach was found to simplify the calibration process and ensure that a reduced sensitivity of the model is achieved.
- As a consequence of increasing computing power and improved detailed hydraulic models, detailed continuous simulation should be used where practical to remove the errors associated with the unit hydrograph modelling of smaller events.
- The observation of sediment trap behaviour highlighted the importance of locating proposed sediment traps at locations where a bed-load mode of transport is prevalent.
- It is recommended that when assessing the maximum particle jump length for a range of particle sizes, the particle with a sedimentation parameter of 10 should be used.

6.4 Recommendations for Further Work

A number of areas where further work is required have been identified. These are summarised below:

- Further field activities are required to extend the scope of long-term data sets of sediment bed levels and sediment trap fill levels
- Improved high resolution data for storm event effects on sediment beds and traps are required to provide further details of the erosion/deposition processes and confirm the assumptions made in this study (see Section 4.8 and Section 4.10).
- Detailed laboratory studies using real drainage sediments are recommended to further observe erosion deposition processes in detail and allow improved definitions of the points of changing transport mode
- It is recommended that the concept of critical shear stress to prevent sediment deposition should be reassessed following the further field and laboratory studies.

- To ease the processing of data when determining the locations of potential deposits, it is recommended that a GIS tool could be developed and combined with the statistical outputs from long-term continuous hydraulic simulation.
- It is recommended that the available methods of sediment prediction should be regularly reassessed to determine the most suitable sediment transport and erosion methods. This assessment should take into account data requirements and the sensitivity of the models.

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Appendix A - List of Publications

Fraser, A. G., Ashley R. M., Sutherland M. M., and Vollertsen J. (1998) Sewer solids management using invert traps. W*at. Sci. Tech.* 37, 1: 139-146.

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Appendix B - Application of InfoWorks 5.0 to Test Catchment

InfoWorks 5.0 was used to test the performance of the existing features of the software on a test catchment exhibiting sediment deposition problems.

The catchment of Lugar is part of the Cumnock drainage area and is located approximately 15 km to the east of Ayr. The catchment covers an area of approximately 24 ha and contains a total population of 221. The catchment has a significant watercourse, the Lugar Water, which flows from north-east to south-west, which in this area has been classed as salmonid and of medium amenity. The upper areas of the catchment are initially steep, draining to a relatively flat trunk sewer. This trunk sewer discharges to a pumping station via a complex arrangement of overflows. This arrangement has been deemed unsatisfactory.

Investigations in the catchment have highlighted the significance of sediment deposits. Deposits are seen to occur in localised areas at the head of the catchment and more significantly, along the length of the trunk sewer. As a result of the large influence that these sediments have on the operation of this small catchment and the availability of good CCTV coverage in the area, a brief study was undertaken to test the capabilities of Infoworks 5.0 sediment transport model in practice.

The format of the test was as follows:

1. Initially, where possible, the user guides were closely followed to allow the model to operate as intended;
2. Following the initial tests, the procedure was modified to see if modelling results could be improved but using only realistic calibration parameters.
3. As all quality models to date have been created using Ackers and White relationships, only this approach was tested.

For the initial tests, default values of sediment characteristics were used along with the default runoff models. As the default modelling procedure is to use only one sediment fraction, no guidance on suitable sediment characteristics for an additional sediment fraction were provided. Consequently realistic parameters were selected from appropriate literature sources (Crabtree, 1989; Ashley et al., 1989; Wotherspoon 1994; Chebbo & Bachoc, 1992; Ristenpart, 1995). The resulting characteristics are given below in Table B.1.

Sediment Fraction	d50 (mm)	S.G.
SF1	0.4	2.0
SF2	0.04	1.7

Table B.1 - Initial testing sediment characteristics

Sewer flow quality models have historically been shown to be extremely sensitive to the selection of initial conditions. In order to have any confidence that the models are representing reality and not operating as a "black-box" simulation, any initial conditions used should also match reality. For sediment transport this means that initial sediment stores (catchment surface and pipe) should be at similar levels and locations to observed values. InfoWorks however does not allow the user to enter the depths and locations of active sediment directly. Consequently, initial depositional patterns must be determined through an initialisation simulation. The advised procedure to establish initial conditions within InfoWorks documentation was followed. In this procedure, 2 options are proposed:

1. *"Carry out an initialisation run to generate initial conditions for the network. This run will probably be a dry weather flow run. The dry weather flow run does not include a rainfall event. All other parameters and inputs must be identical to those of the modelling run."*
2. *"Optionally, you can set the initial mass of sediment on the catchment surface before the start of the simulation."*

From Wallingford Software online documentation 2002.

As no details of the levels of surface sediments were available and the user is not able to set levels of in-pipe active sediment, option 1 was utilised. In accordance with the guidelines, a dry weather period of 6 months was selected in which to deposit sufficient erodible sediment throughout the catchment.

The results of this initial dry weather flow simulation were then compared to the recorded sediment patterns. Figure B.1 shows observed patterns of deposition in the catchment recorded during an extensive CCTV survey. Although not all pipes were surveyed, the observed data show sediment deposits recorded at some of the most upstream locations in the system. For the observed data, only areas with more than 15 mm of sediment are shown. As the pipes in this part in the catchment are generally small (150 mm to 400 mm) a relatively low level of sedimentation was deemed significant in a large portion of the catchment. However, the most extensive area of deposition is found in the trunk sewer.

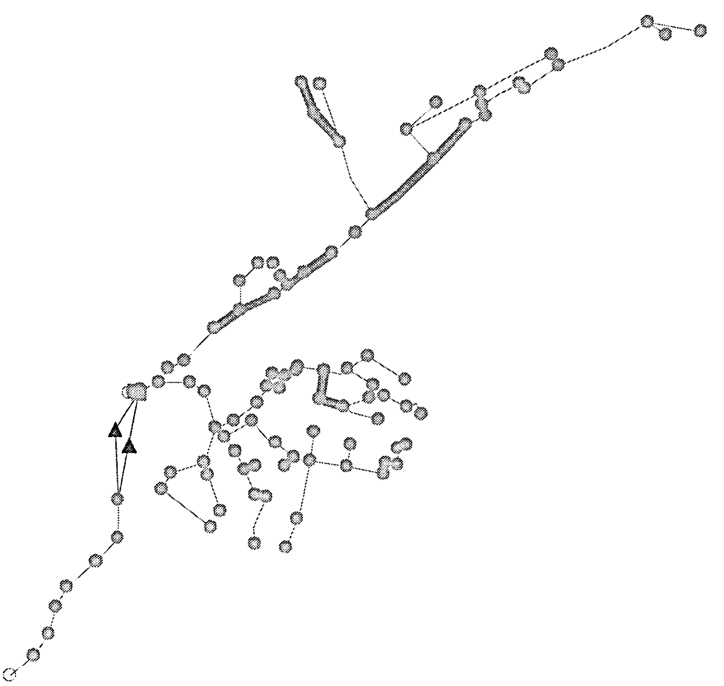

Figure B.1 - Observed deposition in Lugar catchment

Figure B.2 shows the results of using purely dry weather conditions to estimate the extents of deposition. Sediment locations of depth of 5 mm and greater are shown. As can be seen, a relatively low level of deposition is predicted. It was never intended that this limited test should produce actual depths matching those of the observed data set (as the duration of deposition and preceding conditions are not known). However, for the initialisation to be valid, the general extent of the deposition should be replicated. This is not the case.

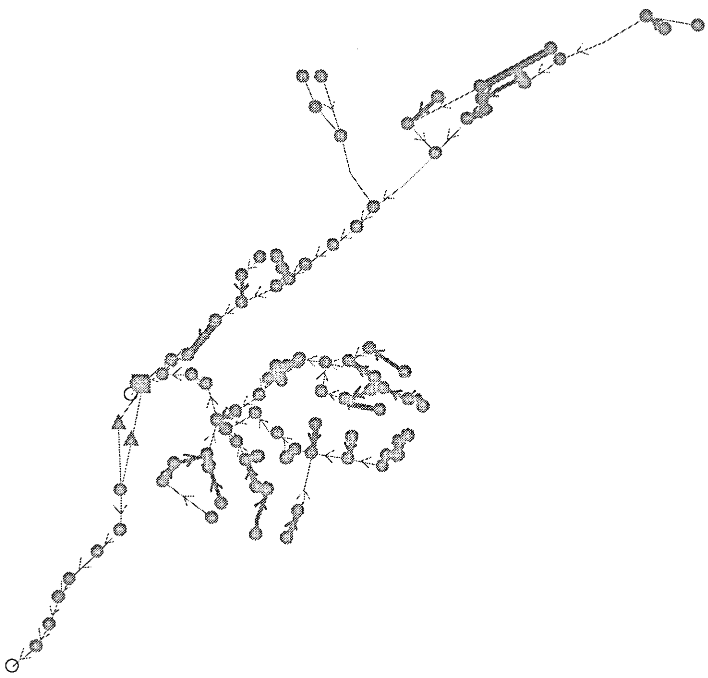

Figure B.2 - Predicted pattern of deposition using InfoWorks initialisation procedure

The principal drawback of the procedure used above is that only dry weather conditions are used. This has two main impacts:

- As only dry weather solids are used (SF2), the locations of deposition are restricted to those for one particle size only. This will result in the under prediction of sediment locations;
- As flows never exceed peak dry weather flow the occasional flushing of pipes does not take place. This results in increased depths of deposition at these

locations. It is believed that this is most significant in pipes of slack gradient near the head of a system.

In reality, sediment deposition involves a complex process of dry weather deposition, storm erosion and storm deposition. These processes interact to produce patterns of deposition which vary with a range of particle sizes at varying locations. In its simplest terms this means that large storm solids will settle out at different locations than more mobile dry weather flow solids. In order to accurately represent depositional patterns (and hence the initial conditions for single event analysis), storm behaviour must therefore be represented.

As the default procedure and values did not produce results comparable to observed values, an improved procedure was sought. It was hypothesised at this stage that the exclusion of storm conditions and solids on the determination of initial conditions was wholly inappropriate. Field observations have indicated storm deposits to play the most significant role in terms of the reduced capacity of pipes. Consequently, the approach was altered to allow a mixture of dry weather and storm conditions to be simulated continuously. This procedure used extensive use of Infoworks 5.0 "dry weather flow simulation mode" in order to reduce simulation times.

Infoworks allows only sediment fraction 1 (SF1) to be washed off during rainfall. Consequently it was assumed that this fraction is intended to represent catchment surface solids. The sediment parameters were selected on the basis of the findings of the literature review carried out for this study, with realistic values selected for dry weather flow sediments (SF2) and storm sediments (SF1). Checks were also made to ensure the compatibility of the hydraulic and quality wash off models.

Sediment Fraction	d50 (mm)	S.G.
SF1	0.4	1.7
SF2	0.05	1.4

Table B.2 - Modified sediment characteristics

Observed rainfall records collected during the Lugar model verification were used as "typical" model inputs. As these records were only in the order of eight weeks, three initialisation model runs were scheduled, each using the final state of the previous run as the input to the next. In this way, six months of continuous simulation were carried out. The resulting patterns of deposition are shown in Figure B.3.

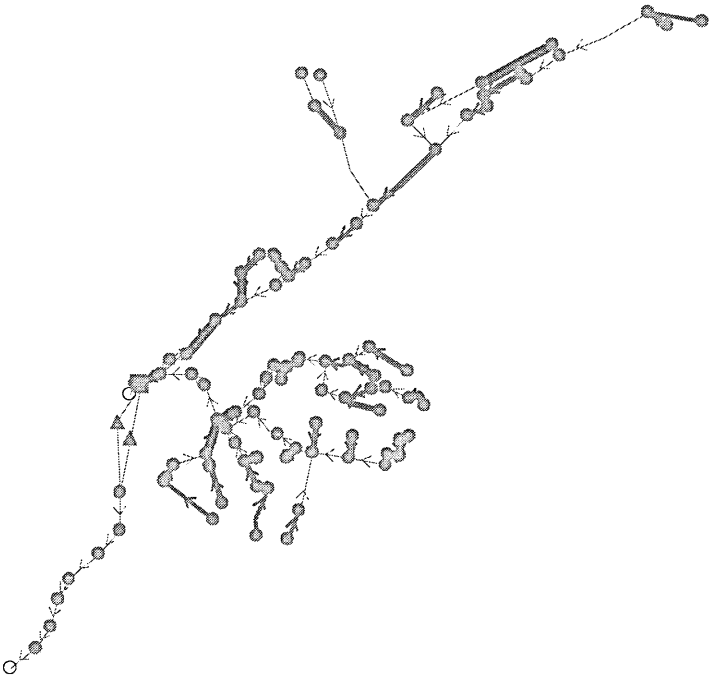

Figure B.3 - Predicted patterns of deposition using storm and dry weather flow inputs

The above figure shows these results plotted to the same scale as used in Figure B.2. As can be seen, the extents of this depositional pattern are far greater with more significant deposition along the length of the trunk sewer and at more upstream locations. These data compare far more favourably to the measured data set. As the CCTV survey was not carried out at all pipe locations it should be expected that the model should predict sedimentation in a greater number of locations. This can be observed to a limited extent to the south east of the catchment.

Although the improved procedure produced better results spatially, the depths of deposition were not directly comparable. No real conclusion on the performance of the quantities of material deposited can be drawn from this exercise, as only a current "snapshot" of deposition was available. However the modelled rates of deposition do not deviate significantly from "typical" figures observed elsewhere. As a refinement for defining initial sediment depth as well as location, "permanent" sediment depths were entered at key locations. These permanent deposits were typically at the points of the most significant deposition and were used to represent the "semi-permanent" deposits often seen to underlie more readily erodible material. In this way, the correct restriction was placed on hydraulics without adversely affecting the depositional processes.

The overall sensitivity of event analysis to the method used to determine the initial conditions was assessed using a design event with a return period of five years. In each case, the same parameters of particle size and density were used in order to make them directly comparable (the modified data set).

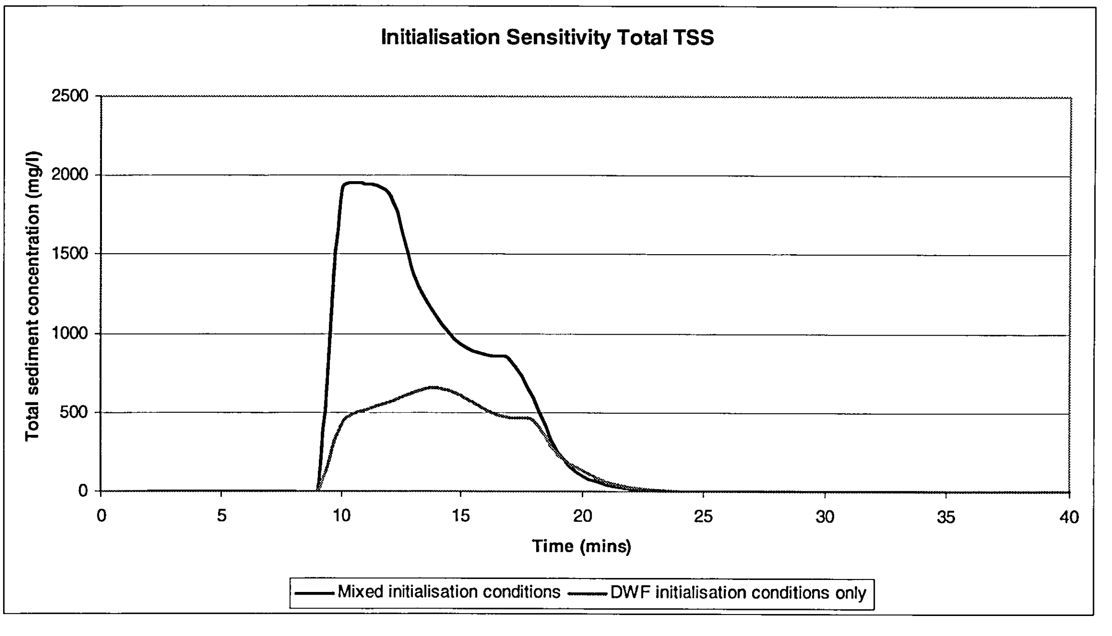

Figure B.4 - Sensitivity of overflow discharges to initialisation method

Figure B.4 shows the effect the chosen method of initialisation has on a potential CSO discharge. As can be seen the more extensive depositional patterns produced by the mixed initialisation condition result in a significantly larger flush (approximately X 3) of total suspended solids. This behaviour is broadly in line with expectations. However, more unexpected results are revealed when examining the outputs in terms of sediment fraction 1 (SF1) and sediment fraction 2 (SF2).

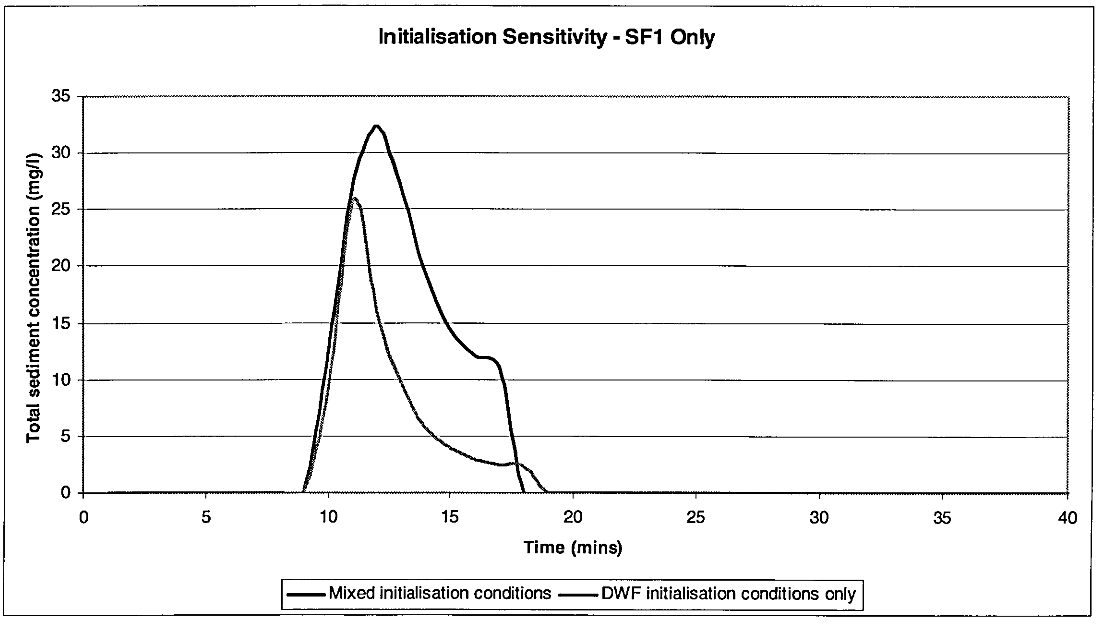

Figure B.5 - Sensitivity of overflow discharges to initialisation method – SF1 only

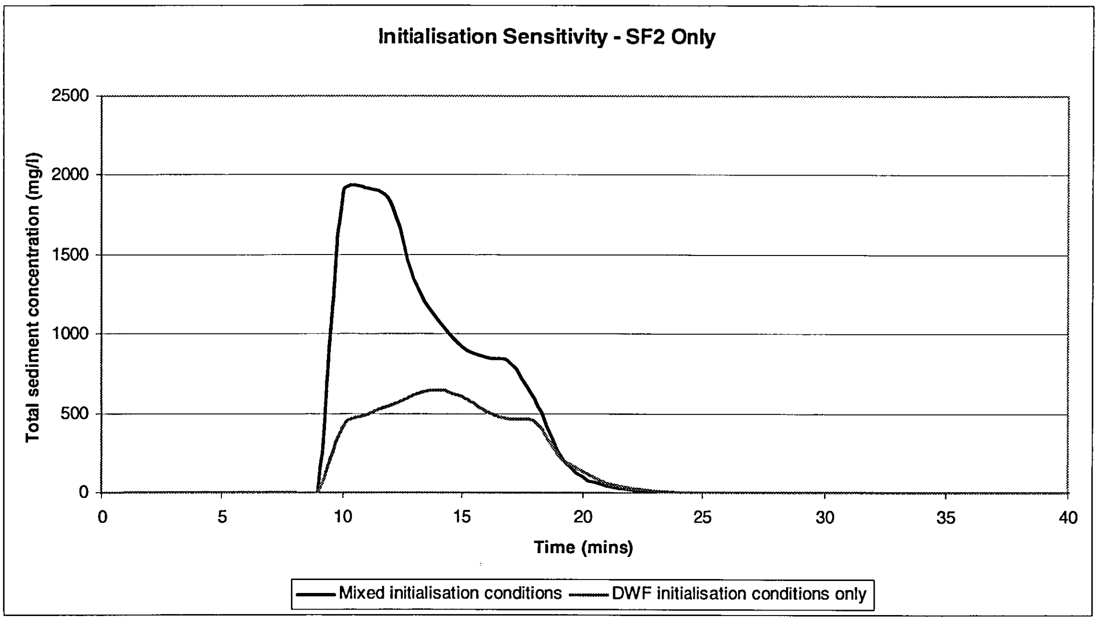

Figure B.6 - Sensitivity of overflow discharges to initialisation method – SF2 only

Figure B.5 and Figure B.6 show the discharged concentrations for SF1 and SF2 respectively. As can be seen, the sensitivity of the chosen initialisation method is significantly greater for SF2 than for SF1. The exact processes involved in this are unclear as it was expected that the inclusion of storm conditions in the initialisation would result in a significantly greater flush of SF1 but not SF2. The reduced impact of SF1 is partly explained through the partitioning of sediment in the overflow chamber, but this does not explain the greater level of SF2 present in the mixed initialisation data set as the sediment fractions were set to be modelled independently.

It is believed that the principal reason for the differences results from the storm solids settling at different locations from the dry weather flow solids. This additional settlement would then have an effect on the hydraulics within these previously clean areas and would encourage the deposition of dry weather material in these areas also. This therefore increases the sensitivity of the model to the inclusion of storm initialisation as both SF1 and SF2 fractions are affected. In addition to this the pattern of deposition can also be seen to have a profound effect on the shape of the discharged flush. In the case of the mixed initialisation conditions the deposits are more concentrated close to the overflow. This results in a faster more pronounced peak concentration. In the case of the dry weather flow initialisation, the more spread out nature of the deposit locations results in a flatter, three tier profile. It is clear from this work that there are a number of processes whose interaction is not fully understood and should therefore be used with caution by practicing engineers.

A number of deficiencies are still seen to exist in InfoWorks 5.0. The most notable of these are:

- Ackers and White transport relationships are not valid at the Infoworks default particle sizes;
- Initial conditions cannot currently be adequately defined;
- Erosion is still assumed to be a purely granular process;
- A number of minor modifications to the transport equations (limiting Dgr and Cv) have resulted in an inconsistent approach not soundly based;
- Model modifications invalidate verification carried out on previous InfoWorks versions;
- Procedure is still essentially a singe particle size approach (although two-phase model reduces this effect).

However, notwithstanding the deficiencies detailed above, the method of operation of the current release of Infoworks has made significant advances over the initial two phase model tested during this study. The initial testing carried out here has suggested that realistic outputs can be achieved using realistic input parameters. It has also been demonstrated that Infoworks can be used to determine likely patterns of deposition. It is believed that the software can be developed further using the findings of this programme of research and other current work. It is therefore recommended that further testing of the new sediment transport models should be carried out along with the inclusion of a cohesive erosion model and particle sorting through the deposition process.

For accurate event simulation, the initial depositional patterns must broadly match those currently observed in the system. It therefore would seem more appropriate to allow the user to directly input the locations and depths of erodible pipe deposits, as during the data collection phase of a model build, these data would be made available. Currently, although depths and locations of deposits can be entered into the model directly, these data play no role within the quality calculations

Through the inclusion of these items and further testing, Infoworks will be more able to predict actual quantities and potential quality (through particle size) of sediment deposition rather than just likely locations.

Outcomes of Research Conclusions

The work carried out as part of this study has had a direct link to industry and has influenced the development of commercially available models. The most significant of these impacts has been the feedback of depositional patterns into the system hydraulics and the development of continuous simulation options.

However, another of the principal objections to the model continues to exist. This involves the use of a granular analysis to predict erosion of cohesive material. Further work carried out here and at the Katholieke Universiteit Leuven has identified further problems with the current software release (Bouteloougier et al., 2002). It is anticipated that this work will continue to refine the model towards a reliable state.

The current model has been tested for the reproduction of depositional patterns in a test catchment and has been found to perform significantly better than previous release versions. It was also found that model performance could be enhanced through the use of improved model initialisation.

Appendix C - Settling Velocity Methods

C.1 Introduction

The laboratory test result of settling velocity is a frequently used determinant in sediment transport analysis. The characteristic is useful as it combines the effects of both the density and size of tested particles. As the definition of the density and size of drainage solids have their own difficulties associated with their measurement, settling velocity results are often used. Consequently, the use of a settling velocity as an important parameter is now widespread in the areas of settling tank design and more recently the numerical modelling of sediment transport.

However, it has become clear in recent years that a number of methods of measuring settling velocity have been developed and that results of the same sample will vary according to the method used (Arthur, 1996).

All methods employ the same basic principal of timing the descent of sampled particles through a column of water. However, two distinct apparatus types have been identified (Ashley et al., 2004):

- Homogeneous methods – where the initial sample is introduced into the settling device in an "original" fully mixed state;
- Top-induced methods where the solids are concentrated prior to being introduced to the top of the device.

Table C-1 shows the range of settling velocity techniques currently employed and also details the characteristics of each method. Within this study, the UFT method was adopted as a laboratory standard as a result of the success of this method in other field studies.

Settling velocity device	Principle of operation	Original design objective	Sample preparation	Liquid in device and sample volume	Settlement distance and column diameter	Known modifications to device and/or procedure
Multi-port method	Homogeneous	Design of settling tanks	Wet sieving 6 mm	Mixed sewage approx. 30–200 litre sample	2000–3700 mm 150-300 mm dia	i. Procedure adapted for comparability studies by CEGEO, Canada ii. Dimensions and procedure modified by Pisano in USA
CERGRENE 1992 Column	Top-induced	Analysis of solids > 50 μm in sewer systems	Wet sieving to separate particles into > and < 50 μm	Water approx. 20 litre sample	1815 mm 50.8 mm dia	Procedure adapted for comparability studies by Aston Uni., UK, by CERGRENE and by CTIA, France
CERGRENE 1992 Andreasen Pipette	Homogeneous	Analysis of solids < 50 μm in sewer systems	Wet sieving to separate particles into > and < 50 μm	Water 20 litre sample	200 mm 50.8 mm dia	Procedure adapted for comparability studies by CERGRENE and by CTIA, France
UFT Device	Top-induced	Analysis of urban runoff, sediment and slime from combined sewer	Settleable solids separated from sample by settlement for 2 hours in an Imhoff Cone	Water 1 litre sample	700 mm5 0 mm dia above cone	i. Procedure adapted for comparability studies by Aston Uni., UK, by CERGRENE and CTIA, France, and by CEGEO, Canada ii. Dimensions and procedure modified by Pisano, in USA
Aston Column	Top-induced	Analysis of both sinker and floater suspended solids fractions in storm sewage	Sinkers and floaters separated and concentrated by standing in column for 3 hours	Sewage liquor 5 litre sample	1750 mm 50 mm dia	i. Used for foul sewage, and ii. larger dia for chemical analysis Aston Uni., UK
CERGRENE 1995 Column	Homogeneous	Rapid analysis of foul and storm sewage	None	Mixed sewage 3 litres	650 mm 70 mm dia	Under development

Table C.1 – Range of widely used settling velocity measurement methods (adapted from Ashley et al., 2004)

C.2 UFT method (Michelbach & Wöhrle, 1992)

The UFT method was developed specifically for the analysis of urban drainage solids. Within this test, a pre-settled sample is introduced at the top of a transparent cylinder. The cylinder is 0.7 m long and has an internal diameter of 50 mm. The base of the cylinder tapers using an Imhoff cone to a sampling point where samples are extracted using a silicon tube. A schematic representation of the apparatus is shown in Figure C.1.

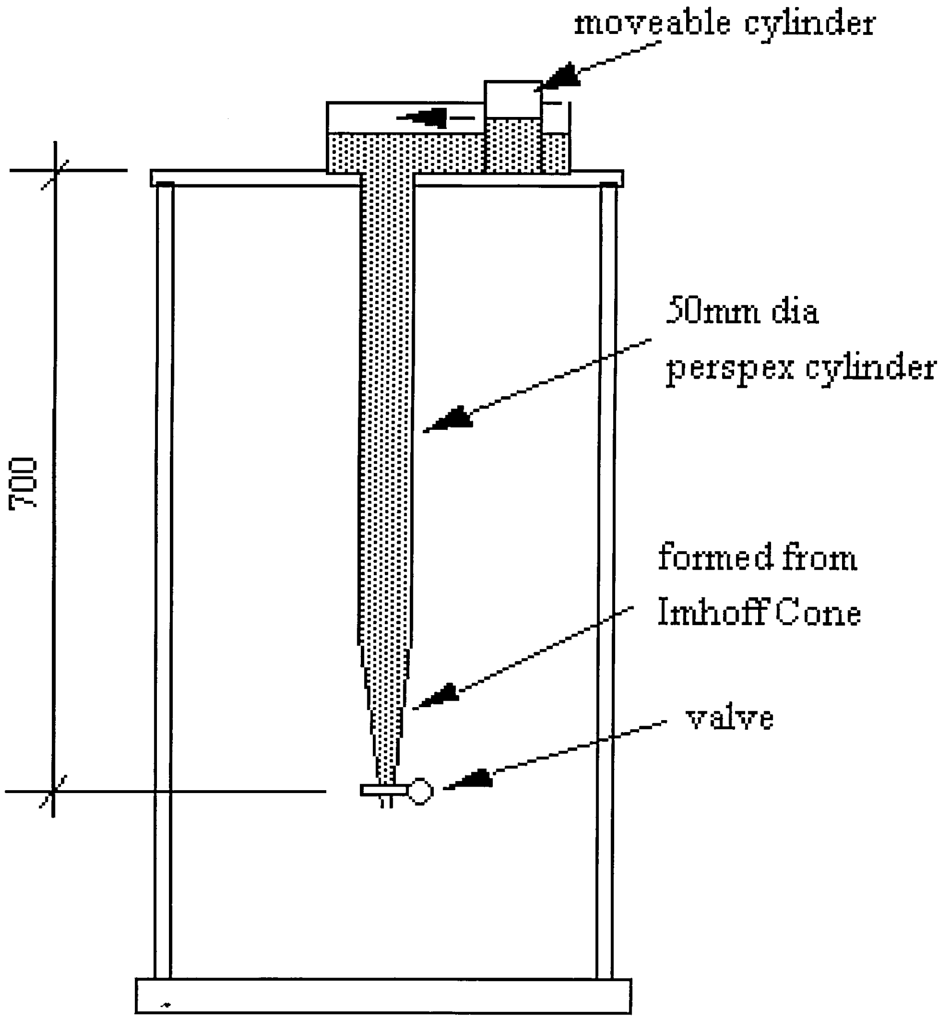

Figure C.1 – UFT settling apparatus

A 20 ml sample is mixed with 75 ml of distilled water. The column is filled with distilled water at a temperature of 20°C to a level of 20 mm above the top of the column (as shown in Figure C.1). A moveable cylinder of the same diameter as the water column is placed in the reservoir area to the side of the column. The diluted sample is then introduced into the moveable cylinder. A sheet of wetted glass is then placed on the top of the moveable cylinder to produce an airtight seal and prevent the rapid mixing of the sample with the main water column as a result of movement. The moveable cylinder is then carefully slid directly over the open end of the settling column and a stopwatch is started.

15 ml samples are then extracted at the base of the cone at time intervals that double at each sample (i.e. 4 seconds, 8", 15", 30", 1minute, 2', 4', etc.). This sampling process is carried out until 2 hours is reached. During this time, care should be taken to prevent settling material from becoming "trapped" against the cone wall. This material should be gently dislodged by tapping on the cylinder wall.

The solids content of each sample is then determined using standard total suspended solids measuring methods and the settling velocity of each sample averaged as 700 mm (fall distance) divided by the settling time.

Finally, a graph of cumulative settled mass versus settling velocity is prepared for the presentation of results. This allows the range of settling velocities and mean settling velocity of a sample to be easily determined.

Appendix D – Trapping Methodology

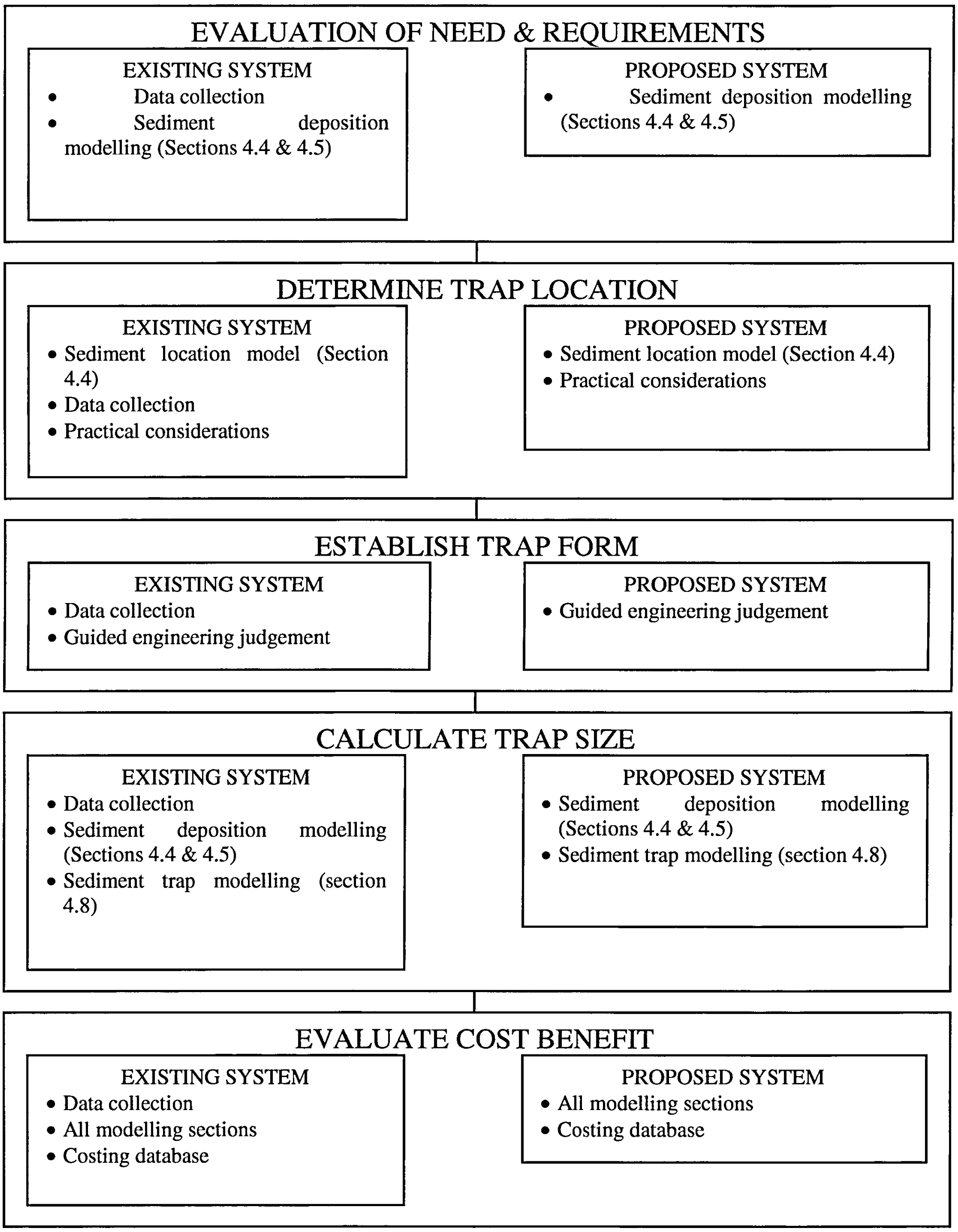

Figure D.1 - Framework for trap application

Figure D.1 sets out a proposed framework of how the methods developed in this study should be applied in the development of sediment management solutions using a trapping approach. Within each principal stage, a source of data or modelling is provided, with reference to a suitable modelling section within this thesis. This is done for the cases of the analysis of an existing system and proposed design.

The following sections describe in more detail, the work required at each of the stages.

Evaluation of Need and Requirements

Prior to the installation of a sediment trap, it is essential that the nature of the sediment related problem is understood. This is an essential first step, as traps may not offer an appropriate solution in all cases. The location of the problem should first be established along with the type of operational difficulties associated with it (e.g. surcharge, flooding, CSO operation, or foul flushes). Following this, the problem material should if possible be characterised through visual inspection and sampling. The sampling of these particles also enables the practitioner to determine the type of particles that should be targeted for removal using the trap. Should these deposits prove to be predominantly organic and the problems dominated by quality effects, trapping sediment may not prove to be a suitable solution. The experiences of this study show that at locations dominated by low-density deposits, rates of trapping cannot be controlled and may result in impractical operational practices (frequent cleaning) (Section 3.6.7). It is instead recommended that the hydraulics of the site and locations downstream should be investigated to determine if any improvements can be implemented. Should this not prove possible, alternative methods of deposition control (e.g. flushing gates or pipe alterations) should be investigated.

In the case of a new system, the only tools that will allow a greater understanding and assessment of any potential problems should be based on the modelling procedures described within this thesis (Section 4.4 and 4.5).

Determine Trap Location

Once the nature of the problem has been understood, the wider context of the problem should be understood through an inspection of the behaviour of the surrounding network. Modelling tools play an important part in this phase of the design process as shear stresses can be thematically mapped throughout the catchment for a variety of conditions and rapidly assessed. Prior to considering potential trap locations, it is essential that a catchment-wide knowledge of sediment deposits is obtained. In the absence of detailed CCTV records, the modelling tools described in Sections 4.4 and 4.5 can be used to identify likely locations and quantities of sediment deposits. It is important (although currently misunderstood), that the trap should not be located at a similar location to that of the sediment problem. It is likely that at these locations, the system hydraulics will encourage deposition, resulting in a trap that fills too quickly with too wide a range of particle types. Instead it is recommended that the trap is located at an upstream location, positioned so as to trap targeted particles whilst they are still in bed-load transit. The removal of these particles will result in improved system hydraulics downstream. Consequently, reduced deposition should take place downstream.

The correct location to trap a targeted particle is a function of the characteristics of the particle and those of the system hydraulics. It is essential that a location be chosen where the targeted particle is moving in a bed-load phase. This is best determined through the calculation of minimum acceptable bed shear for that particle using a sedimentation parameter of 5 (for bed-load transport) and then relating this minimum to the thematic maps of bed shear. In this way appropriate locations can be readily identified. The calculation of the minimum acceptable shear stress can be given by Equation D-1.

$$\tau_o = \rho\left(\frac{w_s}{5\kappa}\right)^2 \qquad \textbf{Equation D-1}$$

Establish Trap Form

The correct trap form appears to be highly dependent on the ambient hydraulics at a given location and the type of particle to be trapped. The findings of this investigation indicate that although partial trap covers may offer some additional selectivity in the trapping of particles, it is their role in terms of preventing the re-suspension of material that is dominant.

Consequently, partial trap covers will provide additional protection to downstream deposition whilst filling, but may fill faster as a result of reduced washout. Provided that granular bed-load transport conditions can be sustained at all times, partial trap covers are likely to offer multiple benefits with selective trapping and improved retention. At sites where near bed solids modes of transport may take place, partial covers will continue to trap this material near the bed and will retain it under storm conditions. The experience of the field activities detailed in Section 3.6.7 indicates that under an open trap configuration, these mobile, trapped deposits are often re-entrained during high flows. In the case of partial covers, these effects may result in a significant proportion of material being trapped that is best dealt with at treatment facilities as a result of its biodegradability. It is therefore not recommended that inappropriate locations be used along with modified trap designs unless it is clear that undesirable material can be excluded.

The findings of the programme of fieldwork associated with this investigation (Section 3.2.7) therefore conclude that partially covered traps are most applicable to sites where traditional bed-load modes of transport are present (see Section 0). It should be noted that this finding contradicts the initial assumption of this study (and related studies), that the partial trap's principal role is the increased selectivity and not increased retention. However, the combination of bed-load transport and trap slot width (partial opening size) allows additional selectivity and increased retention.

Calculate Trap Size

In addition to the determination of the trap location and form, it is proposed that the techniques used in this investigation to model trap filling should be used to determine the suitable storage volume and maintenance regime of a proposed trap. In particular it is recommended that the procedures developed for sediment transport prediction (Section 4.7) and sediment trap filling (Section 4.8) should be used.

If available, hydraulic models can be used to provide flow inputs to the site for dry weather flows and an annual series of storms. These data can then be used to determine bed-load transport rates using the techniques described in Section 4.7. At present, insufficient data exist for the production of generic efficiency curves. Consequently, it is initially proposed for the purposes of maintenance assessment that a worst case scenario of 100% efficiency is assumed. However it is recommended that this efficiency should only be used until the trap is 80% full in the case of open topped traps. At this stage for an open trap, the storage volume should be considered full. In the case of a closed trap the 100% efficiency should be continued until the trap has 95% of its storage volume filled. Initial estimates of trap volume should also be made on the practical basis of land availability, sewer depth and excavation costs.

These efficiencies should be applied to the incoming sediment loads to produce cumulative totals of fill volumes. The time taken to reach the assumed trap full conditions (80% for open trap and 95% for partially covered trap), should then be considered as the design maintenance period. Seasonal effects should be taken into account in order to determine the most appropriate times for trap cleaning. This can be assessed through the observation of predicted filling patterns and carrying out "what-if scenarios" regarding variables such as trap volume and the starting point for the empty trap (e.g. assume spring cleaning). Long-term modelling can be undertaken by combining the modelling procedures of Section 4.7 and Section 4.8.

In this way, an acceptable balance between storage volume and maintenance period can be attained. An investigation into a suitable maintenance period that was acceptable to drainage practitioners has indicated a period of approximately 6-months.

Establish Cost Benefit

Clearly the appropriate maintenance regimes and form of construction are strongly related to available capital and operational costs. Recent trends in the policies of water companies and authorities in the UK have led to the widespread reduction of operational activities. As a consequence of this period of neglect, focus on operational activities and the most efficient way to manage them is once again coming into focus.

The most obvious assessment of the suitability of a trapping method from a financial standpoint is the comparison of the costs involved in trapping sediment to those involved in allowing deposition to take place and then managing the problem reactively through pipe cleaning. However, it should also be noted that a reactive strategy brings with it, increased risks of incurring costs through flooding, increased surcharge and potential legislative implications.

Work carried out as part of this study by Blackwood (Blackwood et al., 2002) has compared the risk and finances of a trapping and reactive strategy using Dundee's interceptor sewer as a test case.

Under this test case, a 6 monthly maintenance period was used for both the trap and pipe cleaning. Comparable volumes of sediment were used, with a minor reduction of the total trapped volume calculated as a result of trap selectivity.

Regime	Annual Maintenance Cost (£)	Cost in Perpetuity (£)	Capital Cost (£)	**Total NPV (£)**
Reactive	8000	160000	0	**160000**
Trap	1430	56600	80000	**136600**

Table D.1 Operating regime comparison

Table D.1 (above) shows a summary of the results of this analysis. This basic cost comparison demonstrates that in addition to operational and risk reduction benefits, a trapping strategy can also offer cost savings. In the case provided above, a total Net Present Value (NPV) saving of £23,400 is evident over a reactive cleaning strategy. In addition to this, risk based analysis is currently being developed to evaluate the wider range of implications of operational strategy beyond purely financial assessments.